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TESTS OF REINFORCED CONCRETE FLAT SLAB STRUCTURES

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AND

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TESTS OF REINFORCED CONCRETE FLAT SLAB STRUCTURES

I. Introduction.

1. *Purpose and Scope.*—It is the purpose of this bulletin to present the results of certain tests made on four reinforced concrete buildings and one reinforced concrete test structure. These tests were made with a view of getting experimental information on the action of the concrete and the reinforcing bars in floor slabs of the flat slab type of building construction. Data were obtained also on the bending action of the supporting columns. Efforts were made to find the distribution of stress in the bands of reinforcement both laterally and longitudinally, and that of the compressive stresses in the concrete on the opposite face of the slab; these in the regions of both the negative bending moment and of the positive bending moment.

The description of the methods used in making the test is limited to those features which are different from the methods used in the tests described in Bulletin No. 64 of the University of Illinois Engineering Experiment Station, "Tests of Reinforced Concrete Buildings Under Load", and from the methods described in a paper on "The Use of the Strain Gage in the Testing of Materials."*

It will be appreciated that the circumstances surrounding the floor test of a building are unfavorable to securing definite and uniform quantitative results. The structure is not homogeneous. There is a distribution of the resistance afforded by the structure to parts beyond the portion which is loaded. Effects of changes in temperature are troublesome. The stresses developed in the steel and in the concrete are small and there is considerable variation between parts which are supposedly similar in action. The conditions under which the measurement of deformation must be made are unfavorable to securing exactness. The location and presence of the loading material also add to the difficulties of the work.

It will be seen that it is impracticable to obtain complete information or to formulate conclusions which are entirely definite. Only general results and conclusions of a qualitative character may be

*Proceedings of the American Society for Testing Materials, vol. 13, p. 1019.

expected. However, it is believed that the tests herein recorded bring out information of value on the action of reinforced concrete flat slabs and of the supporting columns. Since among engineers there is such a marked variation of opinion concerning the action of the flat slab and since there is such uncertainty in the analysis of the flat slab, it is believed that the information given will be regarded as adding to the general knowledge of this subject, and that it will be useful in considering many features of the design of buildings of the type tested.

2. *General Statement of Tests.*—The tests were largely cooperative work. The interests of engineers and contractors in learning the properties of the flat slab and their willingness to bear expenses of the tests made the test work possible. In two cases the tests were made by engineering firms, Mr. Slater being connected with the work, and the data of the tests were placed at the disposal of the Engineering Experiment Station. Mr. Slater was an observer in all of the tests. In every case the data have been carefully worked over in the office of the Engineering Experiment Station and the results discussed and put into form. In the long work of studying the data, many inconsistencies and uncertainties were found, and the presentation of many matters on which it was hoped that the tests would give information had to be abandoned because the results of the tests were indefinite or inconclusive. Results from which at least qualitative conclusions may be drawn have been recorded in the bulletin.

The structures tested and the arrangements for the tests were as follows:

(a) Shredded Wheat Factory, Niagara Falls, N. Y. Flat slab floor with two-way reinforcement. Designed by Corrugated Bar Company, Buffalo, N. Y. Building built by Braas Bros., contractors, Niagara Falls, N. Y. Tested by Corrugated Bar Company.

(b) Soo Line Freight Terminal, Chicago, Ill. Flat slab floor with four-way reinforcement. Designed and built by the Leonard Construction Company, engineers and contractors, Chicago. Tested by cooperation between Leonard Construction Company, Central Terminal Railway Company, and the Engineering Experiment Station of the University of Illinois.

(c) Schulze Baking Company Building, Chicago, Ill. Flat slab floor with four-way reinforcement. Designed by Lieberman and Klein, engineers, Chicago. Built by McLennan Construction Company, contractors, Chicago. Tested by Mr. Slater for American System of Reinforcing. The contractors placed and removed the loading material.

(d) Worcester Slab Test, Worcester, Mass. A sixteen-panel slab having four different designs of reinforcement. Constructed especially for the test. Built according to plans prepared by B. S. Brown, consulting engineer, Boston. Tested by cooperation between Mr. Brown, Worcester Polytechnic Institute, and the Engineering Experiment Station of the University of Illinois.

(e) Curtis-Leger Company Building, Chicago, Ill. Flat slab floor having four-way reinforcement at interior of panel, and two-way reinforcement in region of columns.

Designed by Barton Spider Web System, Chicago, and built by the Simpson Construction Company, contractors, Chicago. Tested by the Engineering Experiment Station of the University of Illinois with the assistance of the engineers and contractors.

Table 1 gives general data concerning the tests.

3. *Acknowledgment.*—The test of the Shredded Wheat Factory Building was conducted by Mr. F. J. Trelease, Chief of the Research Department of the Corrugated Bar Company, and Mr. Slater. The Corrugated Bar Company bore the expense of the test. Acknowledgment is made to this company for the courtesy in permitting the use of the test data.

The technical part of making the test of the Soo Terminal Structure was done as the work of the Engineering Experiment Station. The Leonard Construction Company bore the expense of the preparation for the test and of matters connected with the application of the load. Mr. A. R. Lord, consulting engineer, represented the Leonard Construction Company in planning and carrying out the test. The Central Terminal Railway Company placed the track for the test, and applied and removed the test load. Mr. Slater was in immediate charge of the preparations for the test and of its conduct. The following observers and recorders from the University assisted in the test: Messrs. H. F. Moore, D. A. Abrams, N. E. Ensign, H. F. Gonnerman, H. R. Thomas, G. A. Maney, and M. Abe. Messrs. Meyer, O. R. Erickson, C. J. Nelson, and O. R. Kellner of the force of the Leonard Construction Company also assisted in the test.

The floor test of the Schulze Baking Company Building was conducted by Mr. Slater for the American System of Reinforcing, Chicago. This company, and the contractors, McLennan Construction Company, bore the expense of the test. Acknowledgment is made to the American System of Reinforcing for permission to use the data for publication.

The Worcester slab test structure was conceived and planned by Mr. B. S. Brown, consulting engineer, of Boston, and the expense of carrying out the test was borne principally by Mr. Brown. The materials, labor, and supervision of the construction were furnished by the Allentown Portland Cement Company, Boston, Carnegie Steel Company, Boston, Varnum P. Curtis Gravel Company, Worcester, Aberthaw Construction Company, Boston, and Mr. Brown. The plans for the test were laid out by Mr. Brown, Professor French, of Worcester Polytechnic Institute, and the writers. The conduct of the test was directed by Mr. Slater. Professor H. F. Moore of the Engineering Experiment Station cooperated in the work.

The expense of the Curtis-Leger Building test was borne by F. M. Barton, engineer and architect, Chicago, and the Simpson Construction Company, contractors, Chicago. Mr. Slater conducted the test.

II. The Shredded Wheat Factory Building Test.

4. *The Building.*—The building of the Shredded Wheat Company is of reinforced concrete construction of the flat slab type, three stories in height. The main portion of the building is 265 ft. 4 in. long and 81 ft. wide. The panels are 20 ft. by 22 ft. The floor is the type of flat slab designated by the trade name, Corr-Plate Floor. The floor on which the test load was applied is the first floor above the basement. This floor is nominally 7 in. thick in the central portion of the panel and 9 in. thick throughout an area 8 ft. 6 in. square (the depressed head) surrounding each column. It was designed for 125 lb. per sq. ft. live load. The interior columns have a pyramidal capital of octagonal form 42 in. in diameter at the top and sloping 45° with the horizontal. The slab has two-way reinforcement, designed to resist negative moment at all points across the edges of the panels and positive moment across the center lines of the panels. Corrugated bars were used for reinforcement. Fig. 1 and 2 show the distribution of the reinforcing bars in the test floor of this building. Fig. 2 contains information on the bending and supporting of bars and on other details of the slab. Readings with an engineer's level at numerous positions on the floor gave an average floor thickness of 9.13 in. at the columns and 7.29 in. midway between columns. The average measured depths from the compression surface of the concrete to the center of gravity of the reinforcement of the central panel of the test area were 6.82 in. and 4.95 in. for positions of negative mement in the depressed head and in the thinner portions of the slab

TABLE 1.

GENERAL DATA OF TESTS.

Structure	Type	Test Area		Loading Material	Design Load lb. per sq. ft.	Test Load lb. per sq. ft.	Load Handled tons	No. of Gage Lines	No. of Observers	Days required for	
		In square feet	Per cent of Total Area							Preparation	Test
Shredded Wheat Factory	Two-way reinforcement; depressed head	3960	18.4	Gravel	125	(c) 243	(g) 390	288	2		7
Soo Line Terminal	Four-way reinforcement; depressed head	2304	0.38	Ore cars loaded with broken stone	(b)	700	1006	816	6	13	8
Schulze Baking Company Building	Four-way reinforcement; depressed head	1400	2.95	Brick stacked in piers	300	(d) 722	437	263	1	7	6
Worcester Slab	Four-way reinforcement; no depressed head	3280	100.0	Gravel		(e) 328	459	320	2	11	6
Curtis-Leger Company Building	Four-way reinforcement; no depressed head	1355	11.1	Bags of Cement	200	(f) 500	(h) 237	142	1	7	8

(a) 4-way in central portions of panel, 2-way at column.
(b) Designed for E-50 Wheel-loading.
(c) 191 lb. per sq. ft. over entire 9-panel area.
243 lb. per sq. ft. over only one panel.
(d) 722 lb. per sq. ft. over south two panels, and part of north two panels; 435 lb. per sq. ft. over remainder of 4-panel test area.
(e) 328 lb. per sq. ft. over about 11 panels.
215 lb. per sq. ft. over about 5 panels.
(f) 500 lb. per sq. ft. over 678 sq. ft.
200 lb. per sq. ft. over 1355 sq. ft.
(g) Includes the rehandling of 3 per cent of the material.
(h) Includes the rehandling of 28 per cent of the material.

respectively, and 6.16 in. for positions of positive moment. The columns are octagonal, 25 in. in short diameter for the basement columns, and 22 in. in short diameter for the first story.

The concrete in the building was of excellent quality. Gravel was used as aggregate. At the time of making the test the concrete in the floor was about 80 days old. Four test cylinders, made at the time of pouring the test floor, gave an average strength of about 3500 lb. per sq. in. at an age of 115 days in tests made at the University of Illinois, none falling below 3200 lb. per sq. in.

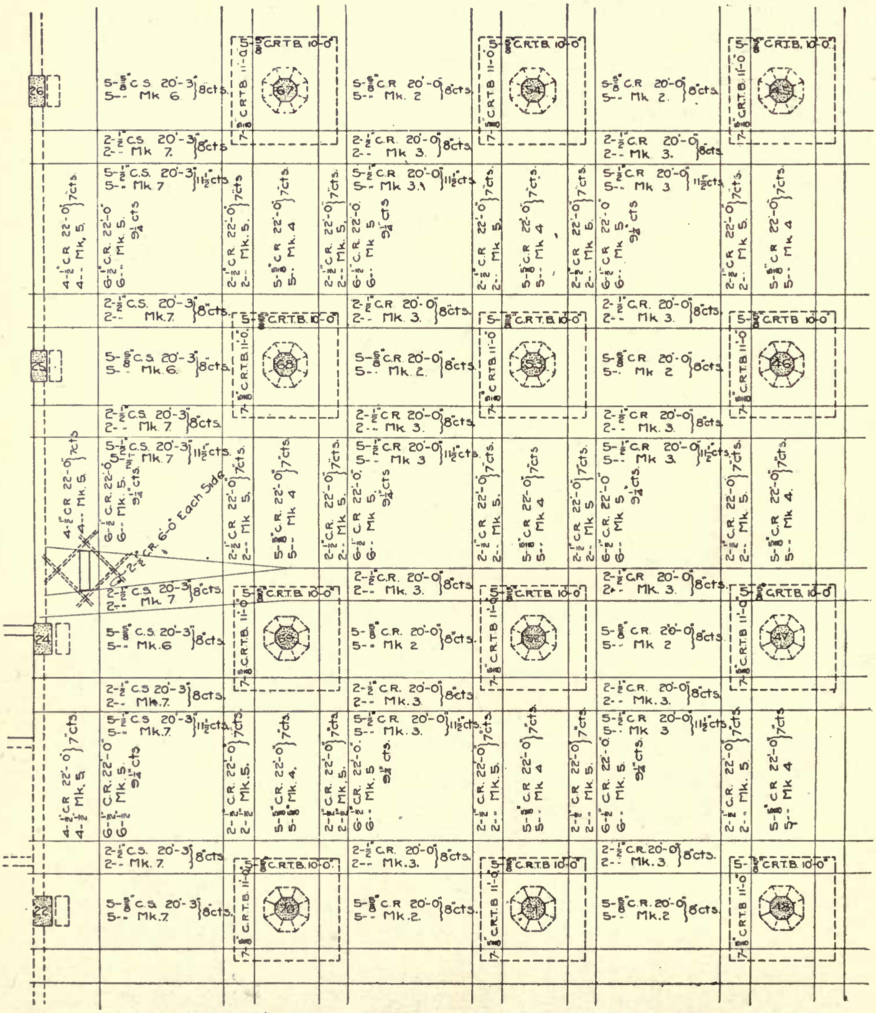
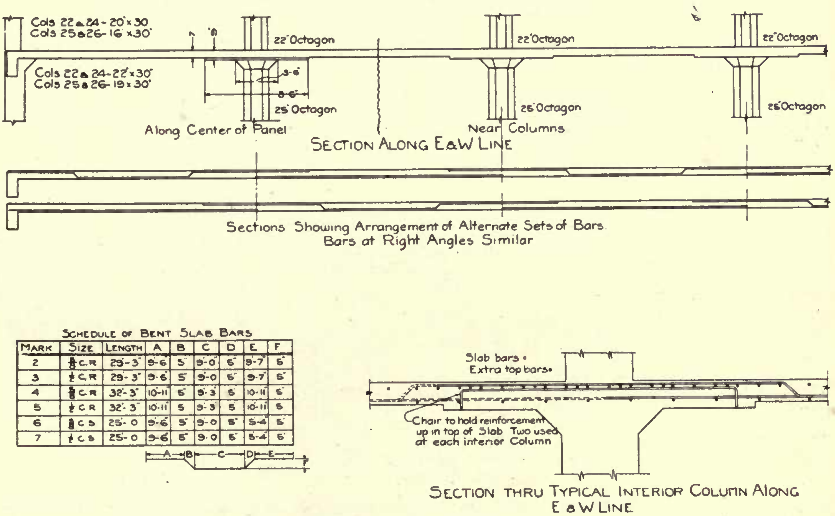


FIG. 1. DISTRIBUTION OF REINFORCEMENT FOR TEST FLOOR OF SHREDDED WHEAT FACTORY.

5. *Testing.*—The deformation measurements were taken on 137 gage lines on the reinforcement and 151 gage lines on the concrete. Fig. 3 and 4 show the location of the gage lines, and Fig. 5 shows the key to the grouping of the gage lines for purposes of comparison. Deflection readings were taken at 33 points (see Fig. 6).

The loaded area is shown in Fig. 6. Gravel was used as the loading material. It was raised by means of a concrete hoist to the second floor and there deposited into the hopper of a concrete chute. By moving the chute the gravel was distributed as desired. Fig. 7 shows the load in position. The load covered the entire panel areas except for aisles 2 ft. wide extending from column to column and boxes about 3½ ft. square placed at the center of each panel. These areas were left uncovered to afford access to the gage lines. Each increment of load was leveled carefully. At the load of 191 lb. per sq. ft., in order to ascertain the total weight per cubic foot of gravel as compacted, a bottomless box was sunk through the gravel after the fashion of an open caisson. The gravel was shoveled out from the inside of the box, measured and weighed. The weight per cubic foot of loose gravel was found to be 113 lb. per cubic foot, and of the compacted gravel, 134½ lb. per cubic foot. As only 79 per cent of the floor area was covered, the corresponding average load per square foot for the total test area was 106½ lb. per ft. of height of compacted gravel. The exact degree of compactness of the gravel at all times was not known, but it is thought that the values used are representative of the test load.



MARK	SIZE	LENGTH	A	B	C	D	E	F
2	5/8 C.R	29'-3"	9'-6"	5"	9'-0"	6"	9'-7"	5"
3	1/2 C.R	29'-3"	9'-6"	5"	9'-0	6"	9'-7"	5"
4	5/8 C R	32'-3"	10'-11"	6"	9'-3"	6	10'-11"	6"
5	1/2 C R	32'-3"	10'-11"	5	9'-3"	5	10'-11"	5
6	5/8 C S	25'-0	9'-6"	5"	9'-0	6"	5'-4"	5"
7	1/2 C S	25'-0	9'-6"	6"	9'-0	6"	5'-4"	6"

FIG. 2. BENDING OF BARS FOR TEST FLOOR OF SHREDDED WHEAT FACTORY.

Nine panels were first loaded (see Fig. 6). Strain gage observations were made at loads of 56, 120, and 191 lb. per sq. ft. The 191-lb. test load was allowed to remain in position two and one-half days. The load was then removed from the three wall panels and from the three panels farthest from the wall, leaving three panels loaded as shown in the figure. Strain gage observations were taken with this load in place. Next, the two outer panels of the three were unloaded and the load on the center panel was increased to 243 lb. per sq. ft.

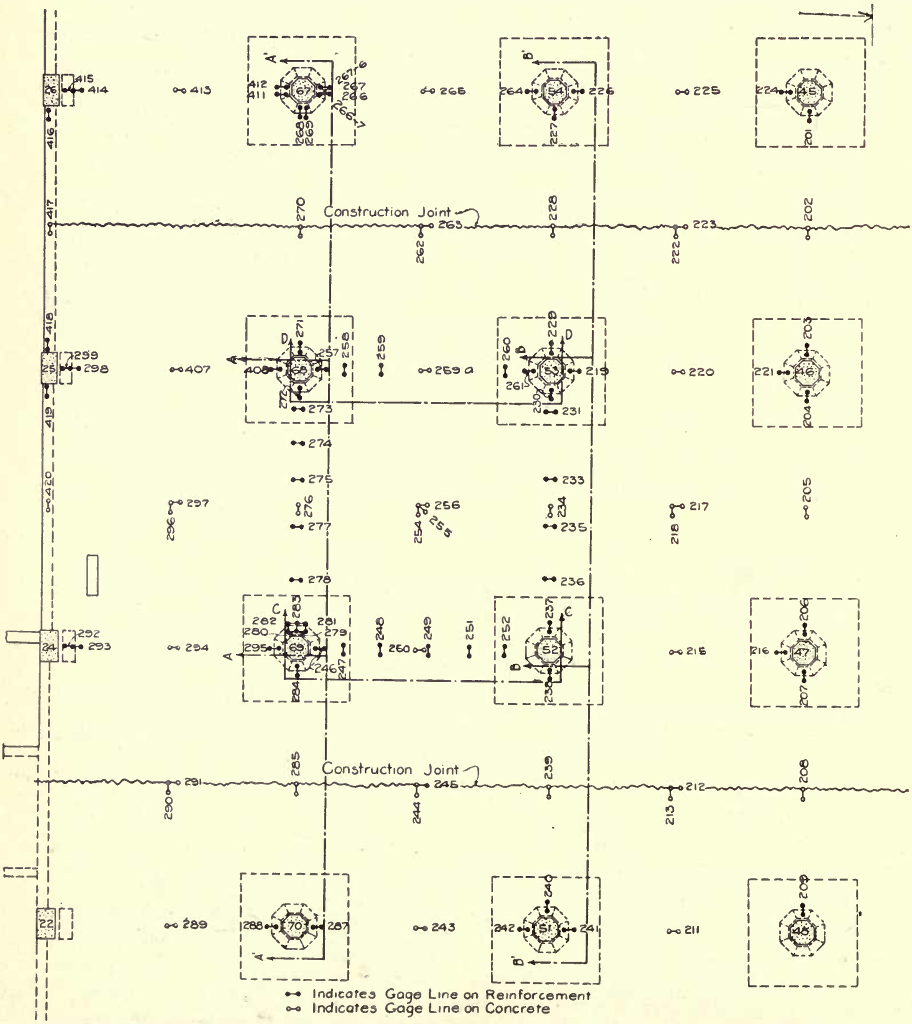


FIG. 3. LOCATION OF GAGE LINES ON UPPER SURFACE OF TEST FLOOR OF SHREDDED WHEAT FACTORY.

At each stage of the test the load was allowed to remain in position at least 12 hours before the final strain gage readings were taken. In order to obtain information on the effect of time on deformation under load, readings were taken at the 191-lb. load immediately after completing the loading operation and again after the load had been in place about 60 hours.

6. *Load-deformation Diagrams.*—The load-deformation diagrams have been plotted in Fig. 8 and 9, the grouping being such as to

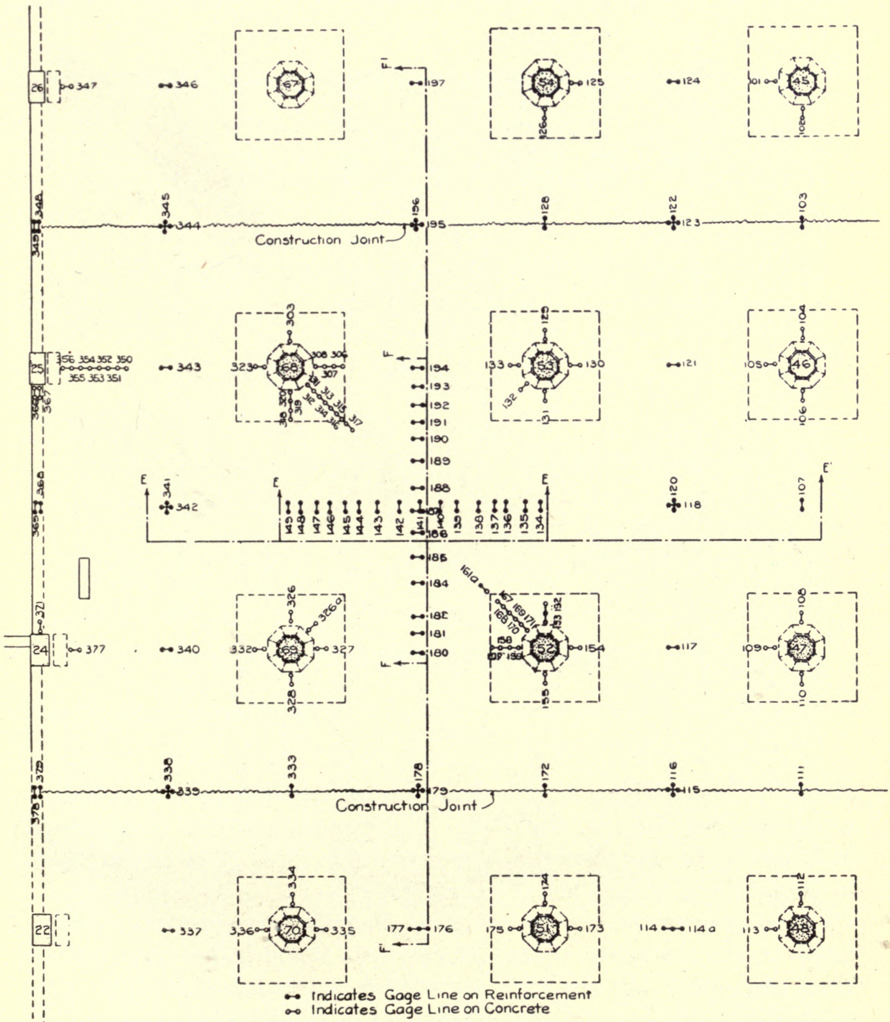


FIG. 4. LOCATION OF GAGE LINES ON UNDER SURFACE OF TEST FLOOR OF SHREDDED WHEAT FACTORY.

place close together the diagrams for gage lines located in similar positions on the floor. In these diagrams the values from the zero load to the first point plotted for the load of 191 lb. per sq. ft. are for nine panels loaded (test area No. 1, Fig. 6). The second point at load of 191 lb. per sq. ft. in each diagram is for three panels loaded (test area No. 2). The point corresponding to 243 lb. per sq. ft. is for the final one-panel load (test area No. 3). The number of panels loaded at various stages of the test is indicated for the diagram marked (1) a, Fig. 8. This is typical of all the diagrams in Fig. 8

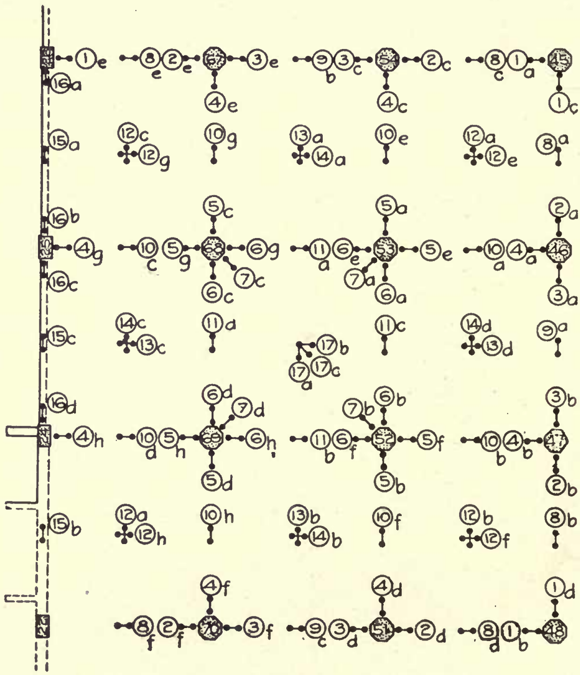


FIG. 5. KEY TO GROUPING OF GAGE LINES; TEST FLOOR OF SHREDDED WHEAT FACTORY.

and 9. The numbers on the diagrams correspond to the gage line numbers given in Fig. 3 and 4. The grouping of the diagrams is indicated by the numbers which correspond to those given in the "Key to Grouping of Gage Lines," (Fig. 5).

In some cases the load-deformation diagrams give peculiar results; however, the similarity of the deformations found in positions remote from each other but similarly situated is marked and tends to give confidence in the results. An example of this is found in Groups 1, 2, and 3 in which, in several instances, slight tension was found where compression would be expected. It is quite possible that these apparently erratic results may be due to general changes in temperature in the concrete.

Examination of these diagrams shows that in all but a small number of cases the removal of load from six panels caused the kind of change in stress which would be expected from the nature of the change in loading.

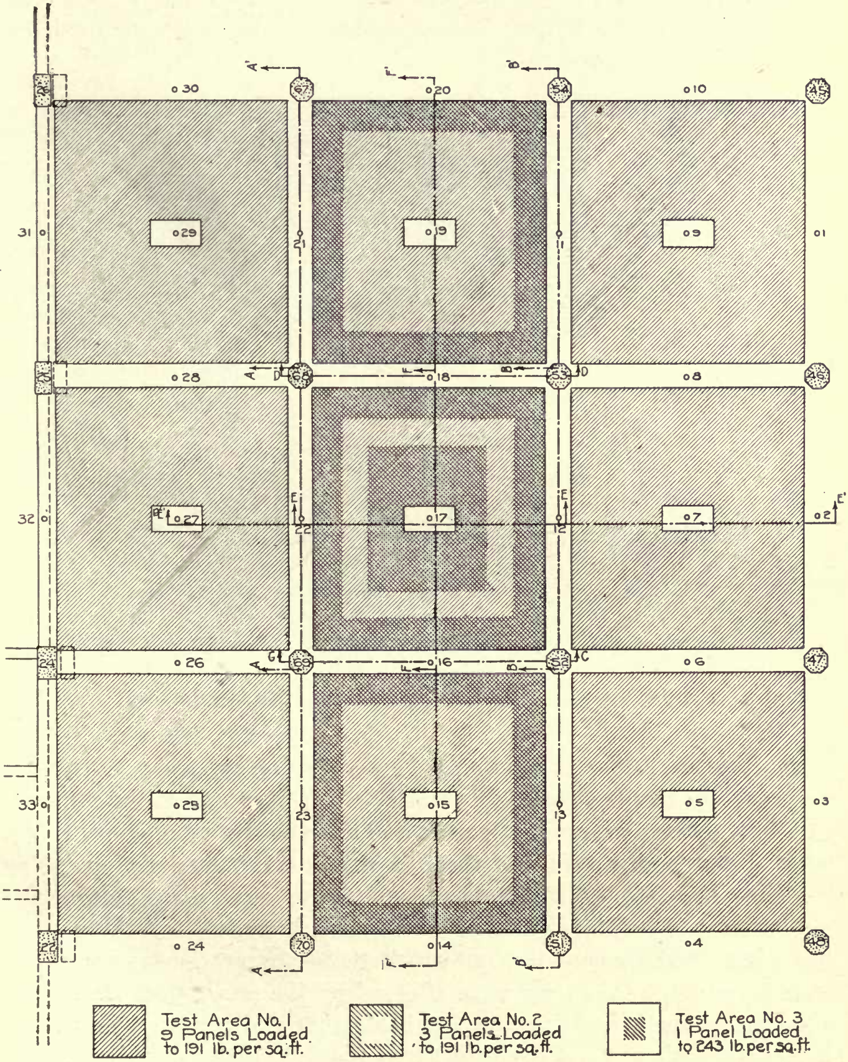


FIG. 6. LOCATION OF DEFLECTION POINTS AND PLAN OF LOAD DISTRIBUTION AT THREE SUCCESSIVE STAGES OF TEST OF SHREDDED WHEAT FACTORY FLOOR.

7. *Effect on Deformations of Standing Under Load.*—With 191 lb. per sq. ft. on the nine-panel area, strain gage readings were taken immediately after placing the load in position and again after it had been in place for about 56 hours. With few exceptions there was a material increase in the deformation during this time, and the increase for the gage lines on the concrete nearly always was greater than for the gage lines on the steel. An examination of 66 gage lines indicates an average increase in unit-deformation approximating 30 per cent for the concrete and 20 per cent for the steel over that which existed

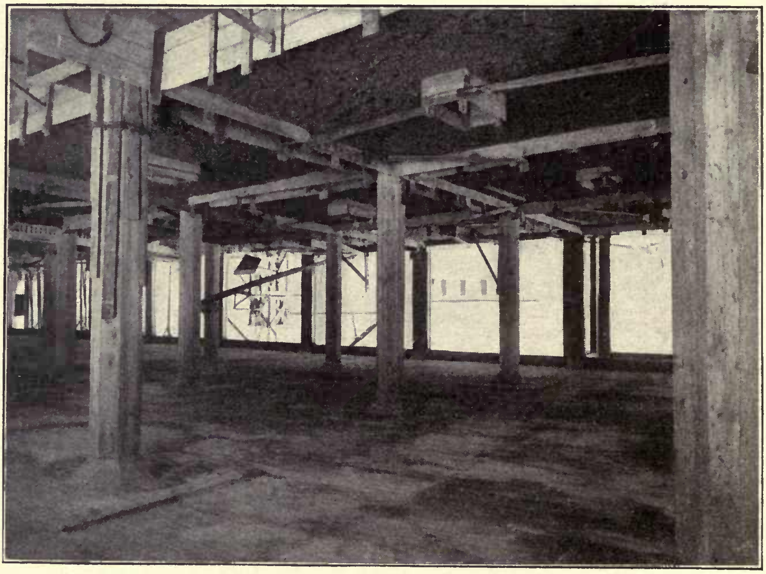


FIG. 7. VIEW OF TEST LOAD IN SHREDDED WHEAT FACTORY.

just after the load was applied. There was no apparent systematic difference in the amount of increase between positions at the columns and positions near centers of panels and none between gage lines at the edge of the loaded area and those in the interior portions.

With the load of 243 lb. per sq. ft. on one panel only, the measurements indicate little or no general increase in deformation, but most of the gage lines on which these observations were taken were so situated that they were affected by the removal of load from the outer panels, and it seems probable that the recovery may have been progressing at the same time that the deformations were increasing due to standing under load.

TABLE 2.

EFFECT ON UNIT-DEFORMATION OF CHANGES IN LOAD DISTRIBUTION.

Plus indicates increase and minus indicates decrease in deformation

Position of Gage lines	Direction of Measurement	Average Change in Unit-deformation in per cent of Unit-deformation before Moving Load	
		Change from 9 panels to 3 panels both at 191 lb. per sq. ft.	Change from 3 panels at 191 lb. per sq. ft. to 1 panel at 243 lb. per sq. ft.
Across center line of panel, Section F′–F′; on steel	E–W	+66	
Across center line of panel, Section F–F; on steel	E–W	+56	0
Across center line of panel, Section F′–F′; on concrete	E–W	+116	
Across center line of panel, Section F–F; on concrete	E–W		—33
Across edges of panel, Sections A′–A′ and B′–B′; on steel	E–W	—19	
Across edges of panel, Sections A–A and B–B; on steel	E–W		+32
Across center line of panel, Section E–E; on steel	N–S	—5	+23
Across center line of panel, Section E–E; on concrete	N–S	—31	
Across edges of panel, Sections C–C and D–D; on steel	N–S	—22	—5

8. *Effect of Change in Number of Panels Loaded.*—Table 2 gives a summary of the results of an examination of the change of unit-deformation caused by the change of loading from nine panels to three panels, and from three panels to one panel (see also Fig. 10 and 11). The areas occupied by these loads and their intensities are indicated as test areas No. 1, No. 2, and No. 3, Fig. 6. In this table increases in the deformation are given a positive sign and decreases are given a negative sign. The most striking features of this table are the great increases in deformation in gage lines which cross the north and south center line of the panel (section F′-F′) in both the concrete and the steel. It is of note also that, in the three instances in this table in which changes in deformation in steel and concrete on the same section can be compared, the changes in concrete are much greater than those in the steel.

The change from nine panels to three panels caused an average increase in deformation of 66 per cent in gage lines which cross section F′-F′. Taking only the gage lines of the middle panel of the three, the average increase was only 56 per cent. (See Fig. 11). The greatest changes of all were in gage lines 179 and 195 both of which lay close to and parallel with construction joints. It is not apparent that the

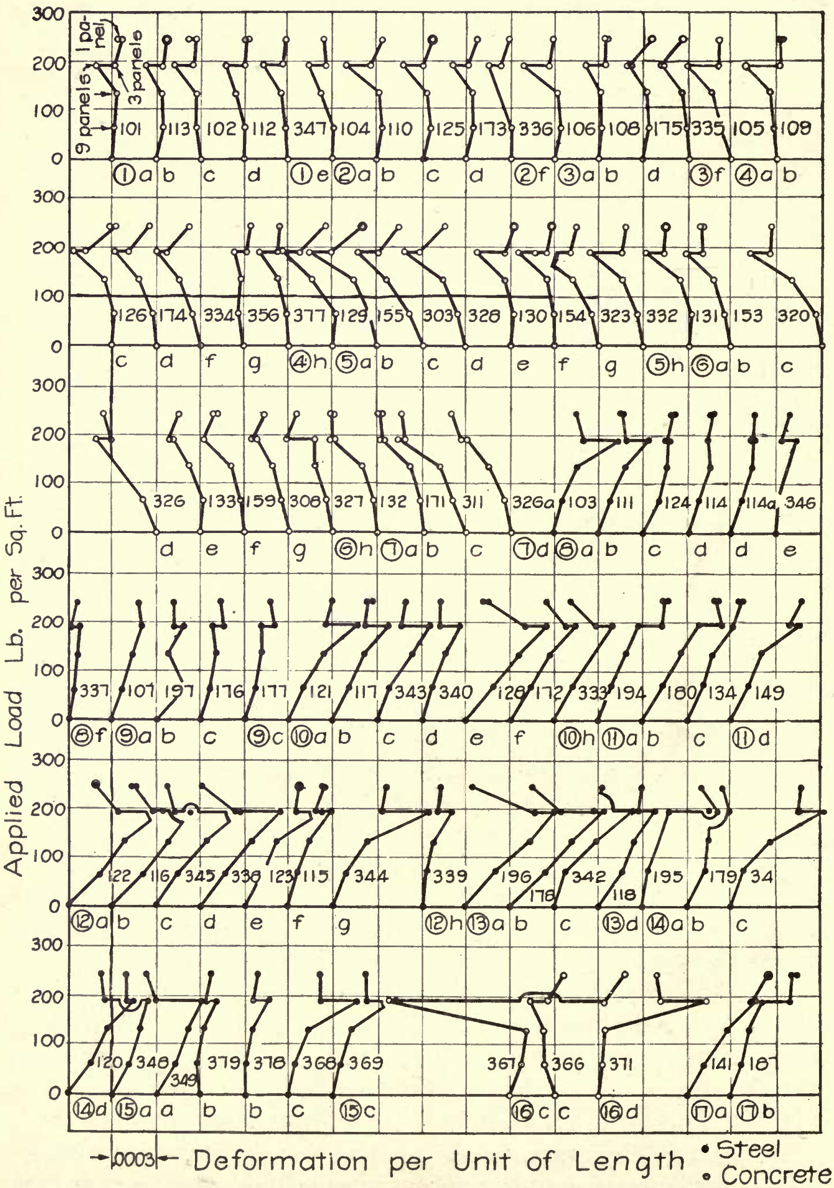


FIG. 8. LOAD-DEFORMATION DIAGRAMS FOR GAGE LINES ON UPPER SURFACE OF TEST FLOOR OF SHREDDED WHEAT FACTORY.

proximity of the construction joint should influence the change since the gage lines do not cross the joint. Gage lines 176 and 177 (at the edge of the test area) showed very large increases also, but that in 197 was below the average.

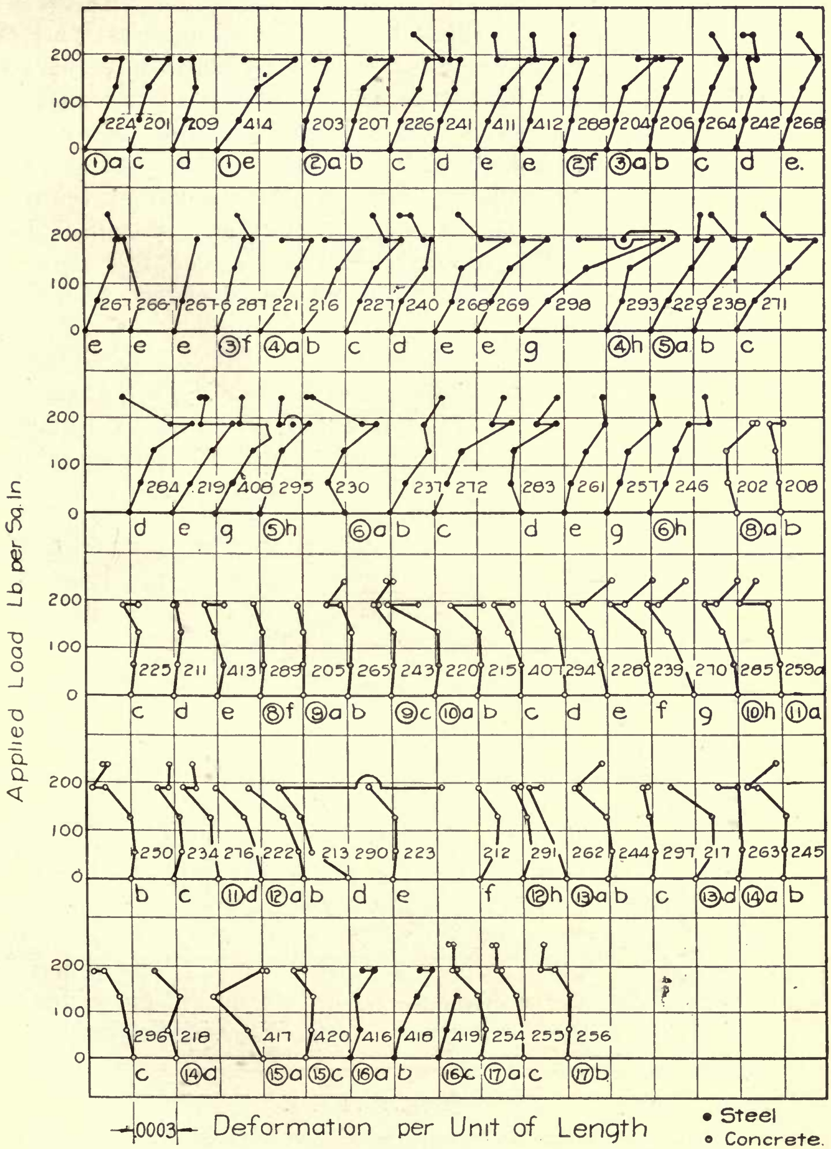


FIG. 9.—LOAD-DEFORMATION DIAGRAMS FOR GAGE LINES ON UNDER SURFACE OF TEST FLOOR OF SHREDDED WHEAT FACTORY.

With few exceptions the deformations in the reinforcement across the edges of the panel (that is, across sections A-A and B-B) decreased with the change of load from nine panels to three panels. These exceptions all are in the case of bars which pass through a column or over a column capital. In all cases the bars which cross the panel edge at points intermediate between the columns lost part of their stress on changing the load from nine panels to three panels. These phenomena would indicate that if, for purposes of comparison, the slab were to be conceived of as a double system of strip beams, the strips passing through the columns could be considered as fixed or nearly fixed at their supports while those passing between columns must be considered to have appreciably less end restraint. The average for all gage lines crossing sections A-A and B-B shows a decrease in deformation of 19 per cent due to the change from nine panels to one panel loaded.

Gage lines were located on the under side of the slab close to the columns in the center panel and in an outer panel. On the removal of the load from the outer panels, the load of 191 lb. per sq. ft. being left on the row of three panels, the deformation in the concrete on the

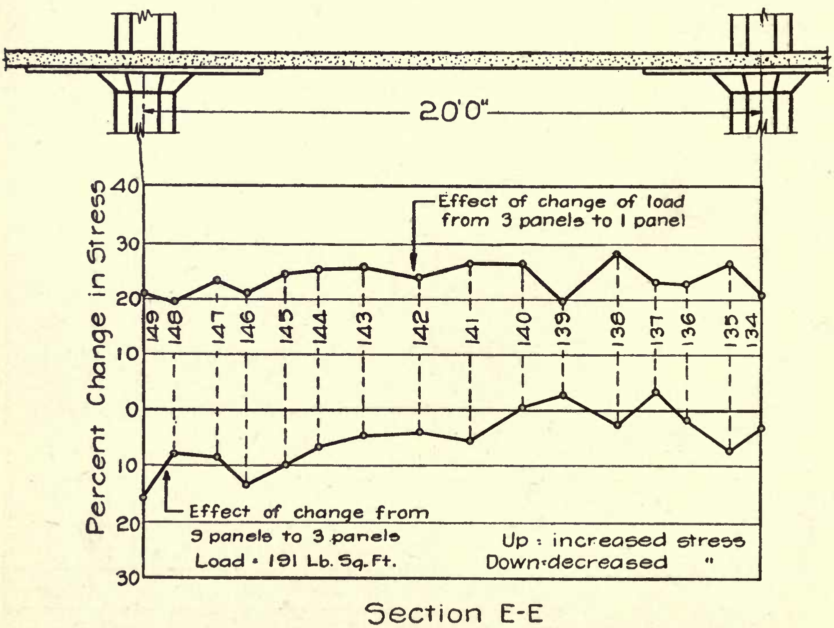


FIG. 10. CHANGE IN STRESS ACCOMPANYING CHANGES IN LOADING FOR GAGE LINES ACROSS EAST-WEST CENTER LINE OF CENTRAL PANEL, SHREDDED WHEAT FACTORY.

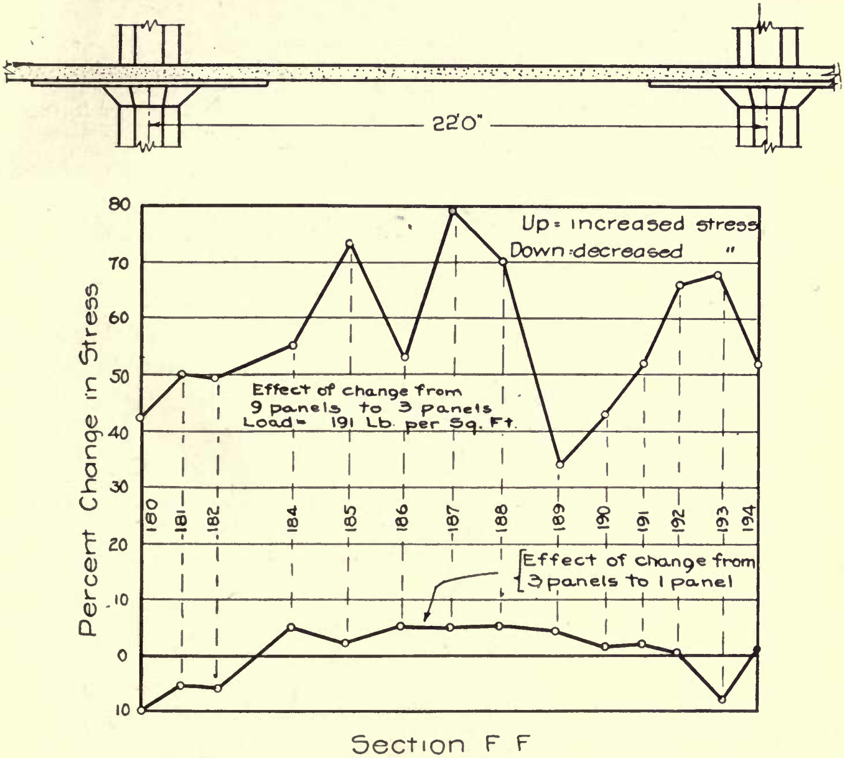


FIG. 11. CHANGE IN STRESS ACCOMPANYING CHANGES IN LOADING FOR GAGE LINES ACROSS NORTH-SOUTH CENTER LINE OF CENTRAL PANEL, SHREDDED WHEAT FACTORY.

under side of the slab at the unloaded side of the column (gage lines 323, 332, 154, and 130) decreased markedly, and the deformation in the concrete on the loaded side of the column (gage lines 327, 159, 133, and 308) increased somewhat. Assuming a modulus of elasticity of 3,000,000 lb. per sq. in. for the concrete, the amount of the compressive stress in the concrete on the loaded side amounted to about 800 lb. per. sq. in., and on the unloaded side to about 300 lb. per. sq. in.

Fig. 13 and 15 show the deformations in the gage lines which cross the center lines of the central panel (sections F-F and E-E) for the three panels loaded and for the one panel loaded. Fig. 10 and 11 show the change in stress in these sections caused by the changes in loading in per cent of the stress present before moving the load.

The decrease in the deformation in bars lying under the load but near the panel edge was more marked than that in bars parallel to them under the central part of the load. Taking into account the

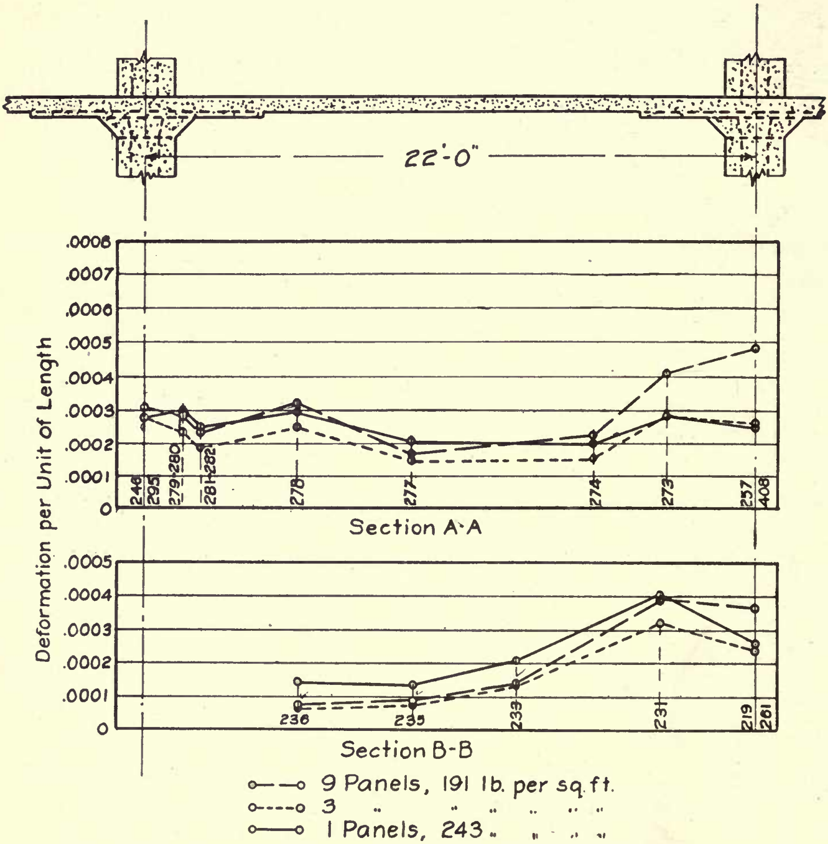


FIG. 12. NORTH-SOUTH DISTRIBUTION OF DEFORMATION IN EAST-WEST BARS ACROSS EDGE OF CENTRAL TEST PANEL IN SHREDDED WHEAT FACTORY.

increase in the intensity of the load simultaneously with the removal of load from the two outer panels, leaving only the central panel loaded, it seems that there must have been a distribution of stress laterally to bars outside the loaded area (or assistance given by the development of stress in the reinforcement in the unloaded panels adjacent to the loaded panel). The effect seems to have disappeared at a distance of about one-quarter panel width from the edge of the loaded area.

For some reasons it might be expected that the change from the three-panel load to the one-panel load would cause an increase in deformations on gage lines which cross section E-E (see Fig. 10). However, the increase in deformation on section E-E (23 per cent)

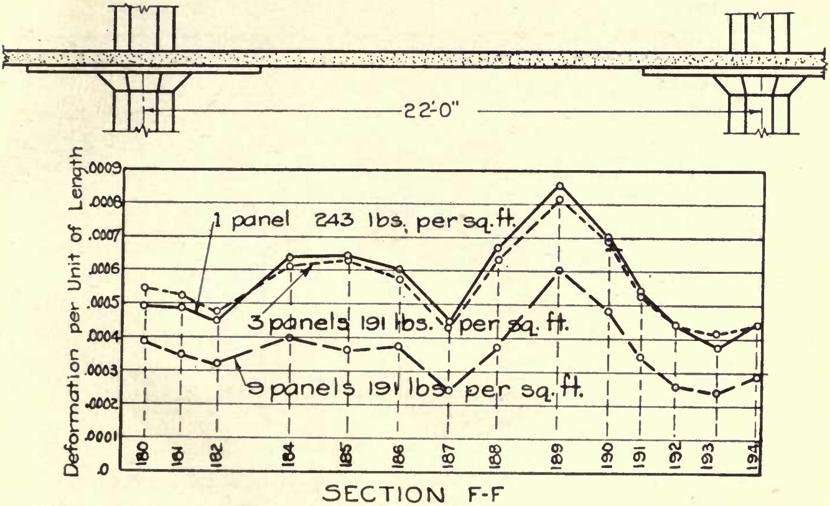


FIG. 13. NORTH-SOUTH DISTRIBUTION OF DEFORMATION IN EAST-WEST BARS ACROSS CENTER LINE OF CENTRAL TEST PANEL IN SHREDDED WHEAT FACTORY.

following the change from three panels at 191 lb. per sq. ft. to one panel at 243 lb. per sq. ft. was less than the increase in intensity of the load (27 per cent).

9. *Distribution of Stress and of Moments.*—In the design of the floor slab the reinforcing bars were distributed across the panel width in accordance with a method which the Corrugated Bar Company derived from the test made by Mr. Trelease on a small rubber slab.* To show the distribution of stress over the width of the slab, as measured in the test, the deformations developed in the reinforcing bars at the load of 191 lb. per sq. ft. over nine panels have been plotted in Fig. 12, 13, 14, and 15, using deformations as ordinates and distances at right angles to the direction of the length of bar as abscissas (width of slab). It will be seen that rather large differences were found in the deformations in bars which were close together. That these differences are not generally errors of observation is indicated by the fact that a set of check readings on a large number of gage lines gave stresses practically identical with those of previous observations at the same load.

Fig. 16 gives moment factors showing the distribution of the positive resisting moment across the width of the slab based upon the measured deformations in the reinforcing bars, and Fig. 17 gives similar

*See Proceedings National Association of Cement Users, vol. VIII., p. 218.

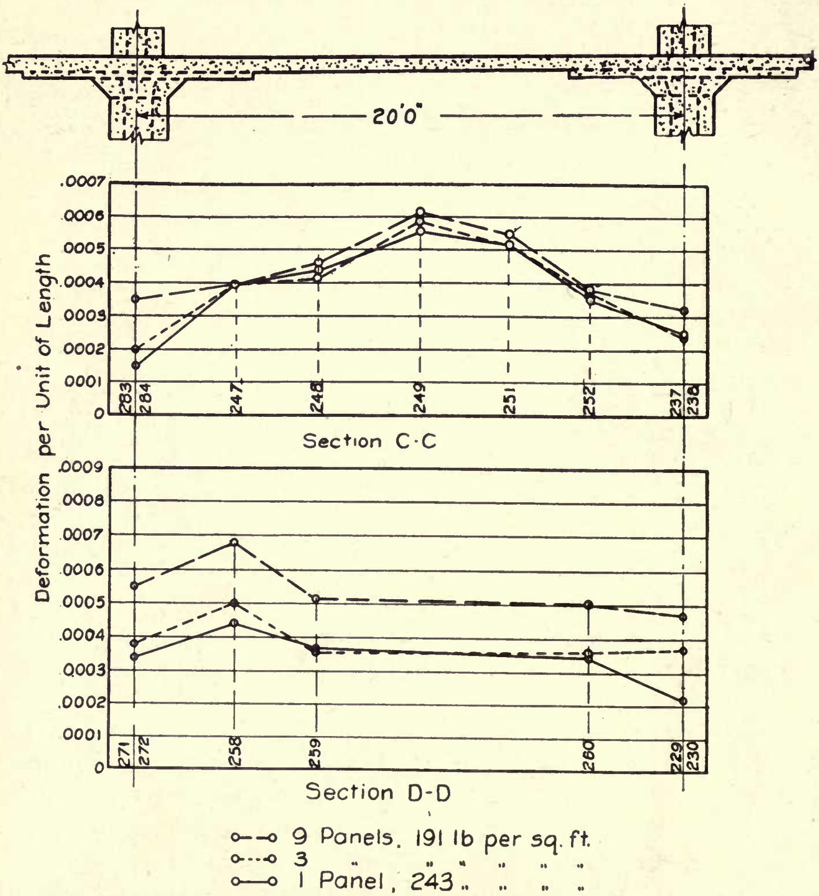


FIG. 14. EAST-WEST DISTRIBUTION OF DEFORMATION IN NORTH-SOUTH BARS ACROSS EDGE OF CENTRAL TEST PANEL IN SHREDDED WHEAT FACTORY.

moment factors for the negative resisting moment based upon the measured deformations in the steel. These moment factors represent the coefficients by which wl^2 must be multiplied to obtain the positive (or negative) resisting moment per unit of width of section developed by the stress found in the steel, w being the load per unit of area and l the panel length center to center of columns in the direction of the stress considered.

In making up these two diagrams the width of the slab was divided for convenience into several portions, and for each portion the average stress in the reinforcement was multiplied by the effective moment arm times the cross-sectional area of the reinforcement. The moments for the several portions were divided by the

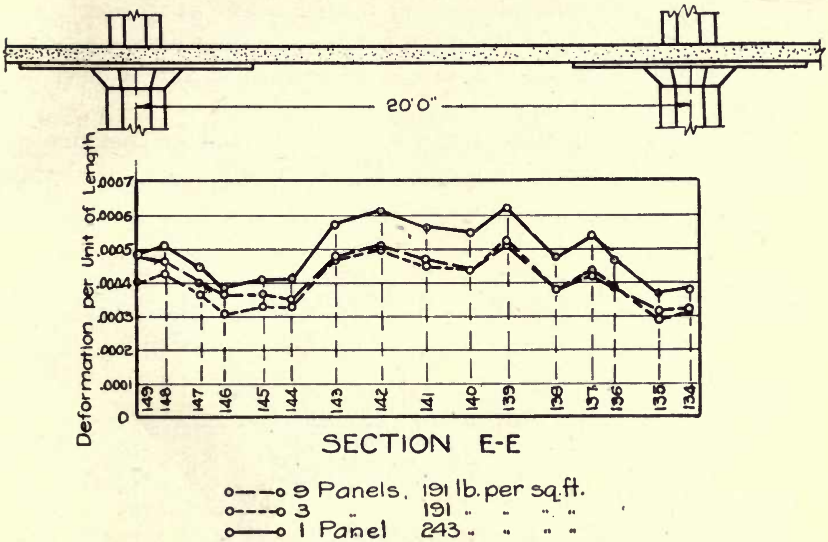


FIG. 15. EAST-WEST DISTRIBUTION OF DEFORMATION IN NORTH-SOUTH BARS ACROSS CENTER LINE OF CENTRAL TEST PANEL IN SHREDDED WHEAT FACTORY.

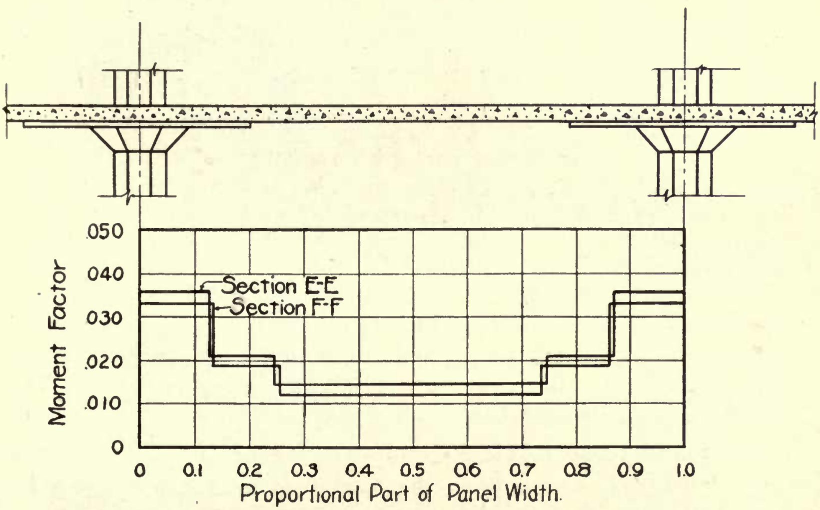


FIG. 16. MOMENT FACTOR DIAGRAM FOR SECTIONS OF POSITIVE MOMENT (SECTIONS E-E AND F-F) OF CENTRAL TEST PANEL, SHREDDED WHEAT FACTORY.

widths and by the quantity wl^2. Calculations were thus made for two sections (E-E and F-F) at positions of maximum positive moment and for four sections (A-A to D-D) at positions of negative moment. The stresses used were taken from Fig. 12, 13, 14, and 15 for the load of 191 lb. per sq. ft. over nine panels in such a way as seemed best to represent the stresses over the entire section. In some cases a uniform stress represented the distribution as accurately as any symmetrical curve which could be used.

It will be noted that the distribution of the values of the moment factors in Fig. 16 and 17 is largely dependent upon the distribution

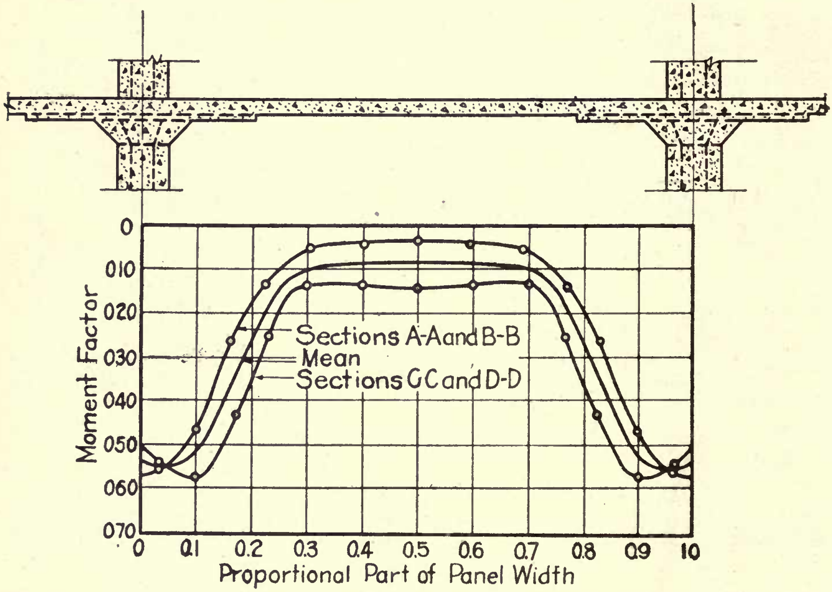


FIG. 17. MOMENT FACTOR DIAGRAM FOR SECTIONS OF NEGATIVE MOMENT (SECTIONS A-A TO D-D), SHREDDED WHEAT FACTORY.

of the reinforcing bars over the width of the section. If the bars had been grouped less closely over the column head there probably would have been less variation in the values of the moment factors.

By integration of the areas between the moment factor curve and the axis, the value for the positive resisting moment due to the measured stresses in the steel for a width equal to the width of the panel is found to be 0.021 Wl and that for the negative resisting moment 0.026 Wl, where W is the total load on the panel and l is panel length. These moments are the averages of the values obtained from the two curves for the positive moment and from the two curves for negative moment. These moments represent the value of the resisting moment developed by the steel for one direction only, as determined by average unit-deformations over 8-in. gage lengths. With the increased deformations at certain places when only three panels were loaded the average positive moment would be found to be larger than the value given here. As the tensile resistance of the concrete may be expected to be considerable for beams or slabs having as low percentages of reinforcement as these and as low deformations as were found in the steel, the moments given above can not be taken to be the actual resisting moments developed in the slab.

10. *Stresses and Moments in Columns.*—In case not all of the panels are loaded, the bending moment developed in the slab at the

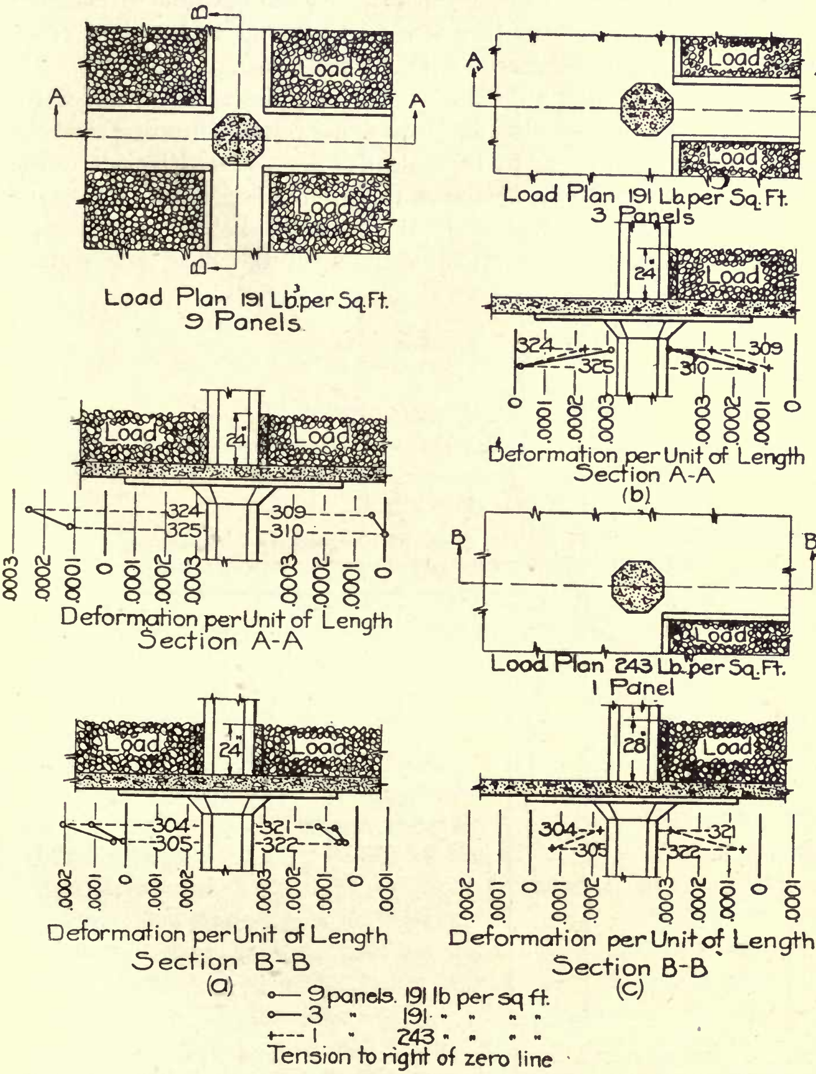


FIG. 18. DEFORMATIONS IN INTERIOR COLUMN NO. 68 OF SHREDDED WHEAT FACTORY.

edge of the loaded panels is resisted by a restraining moment taken by the columns at the edge of the loaded area and by the slab of the adjacent unloaded area. The division of this restraining moment into the three restraining moments, that taken by the column above the floor, that taken by the column below the floor, and that taken by the adjacent unloaded slab, is dependent upon the relative stiffness of these members (represented by the moments of inertia of the members and by the relative length of the members). As the modulus of elasticity of the concrete in the structure is not known and as the effect of the tensile strength and stiffness of the concrete is quite uncertain, it is not practicable to make a definite statement concerning the amount of bending stresses in the columns or the exact relative bending moments taken by the columns and by the unloaded floor. It may be interesting, however, to note the phenomena as they were observed (see Fig. 18).

Gage lines 309, 324, 304, and 321 were located on the four sides of a basement column just below the column capital. The deformations

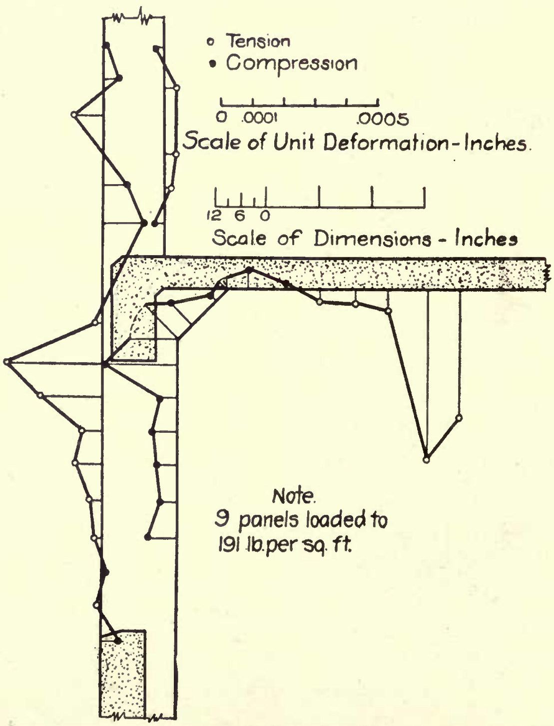


Fig. 19. Deformations in Wall Column No. 25 and Point of Zero Unit-deformation on Under Surface of Test Floor near Wall Column, Shredded Wheat Factory.

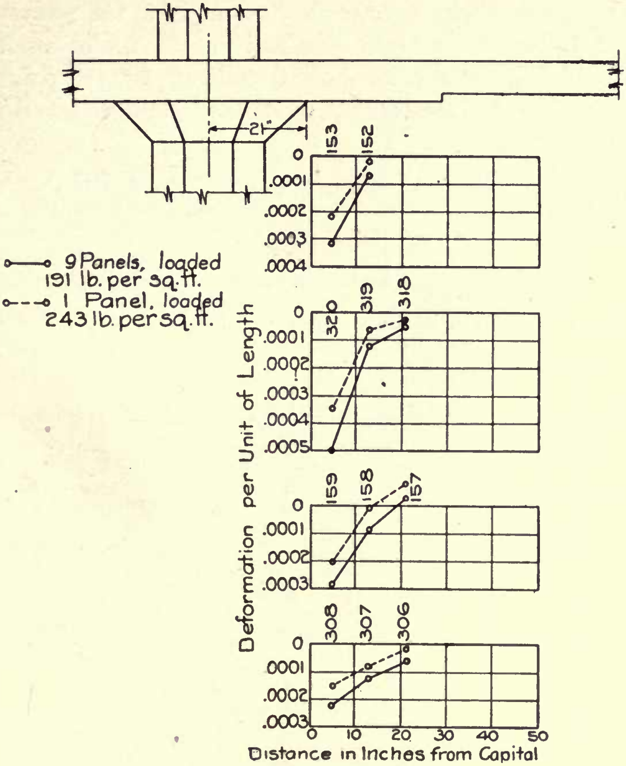


FIG. 20. POINTS OF ZERO UNIT-DEFORMATION ON UNDER SURFACE OF TEST FLOOR NEAR INTERIOR COLUMNS OF SHREDDED WHEAT FACTORY.

in this column show (see Fig. 18) that the partial loading of the slab developed a severe bending in the columns of the basement story. No measurements were taken on the columns of the first story.

11. *Point of Inflection.*—With a view of determining the position of the point of inflection, several series of measurements of deformations in the concrete on the under side of the slab were taken. The results are shown in Fig. 19, 20, and 21. Reference to Fig. 4 will show the location of gage lines. In Fig. 20 only one series of measurements is carried far enough to cover a change from compression to tension. The tendency of these curves to change their slope abruptly makes it impracticable to find the point of zero stress by producing the line till it crosses the axis. Some floor tests have shown the position of zero-deformation of the under surface of the slab to be closer to the column than that for the upper surface, while other tests are quite the opposite. No reason is known for such variation in

results. It seems unwise to base conclusions as to the positions of the point of inflection upon data obtained on the under surface only. The curves in Fig. 20 and 21 show a tendency for the point of zero-stress to move toward the column when the load is changed from test area No. 1 to test area No. 2.

12. *Lintel Beams.*—During the progress of the tests, minute cracks were found in the concrete in various places in the structure,

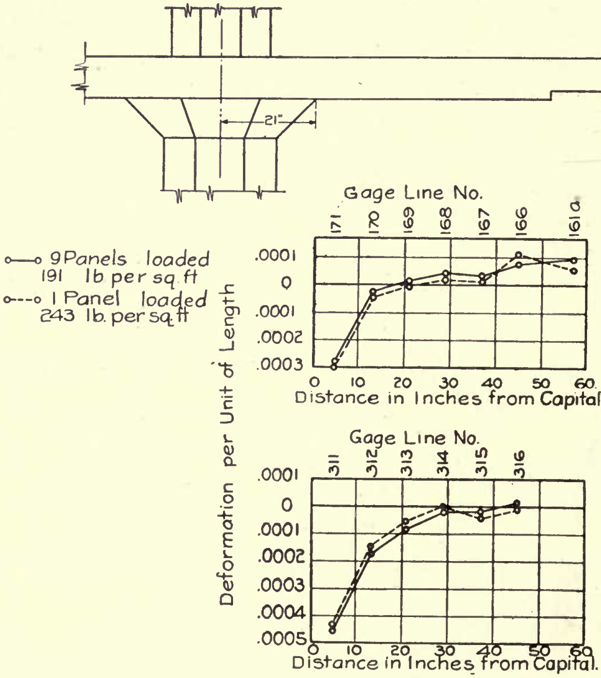


FIG. 21. POINTS OF ZERO UNIT-DEFORMATION ON UNDER SURFACE OF TEST FLOOR NEAR INTERIOR COLUMNS OF SHREDDED WHEAT FACTORY.

and the presence of these cracks confirmed the results of the strain gage measurements. The most important indications of this kind were cracks which were found on the interior side of the lintel beams near their ends. The cracks did not appear on the exterior of these beams and except for this, the cracks, by their position and direction, resembled the diagonal tension cracks ordinarily found in beams subjected to high shearing stress. The web reinforcement in the lintel beams consisted of both inclined members and vertical members which were wrapped about the horizontal bars. The presence of cracks on the interior surface and the absence of cracks on the exterior surface may indicate that this phenomenon was due to torsion. The slab and the lintel beams were built monolithically, and the bending moment developed in the slab at its end by the load on the wall panel would produce torsion in the beam. The formation of the cracks shows that stresses exist in the lintel beam which are not ordinarily considered. The effect of the bending of the slab on the lintel beam to which it is attached should be considered.

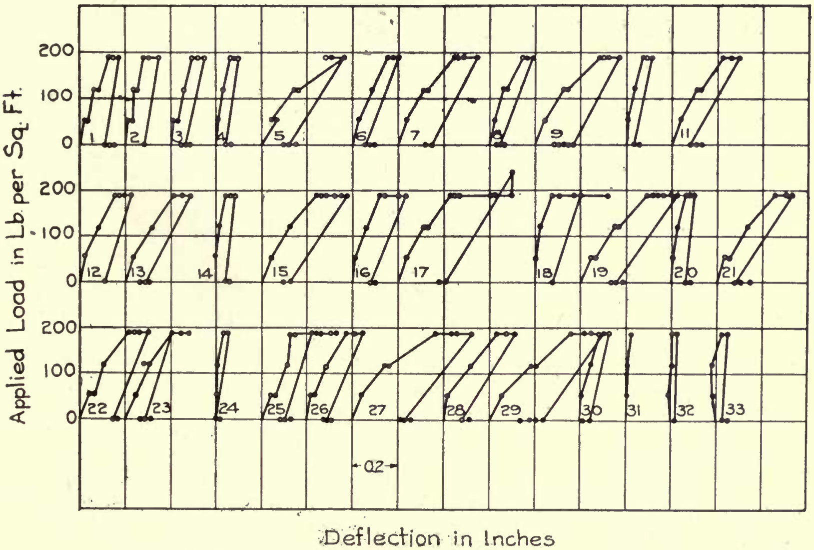


FIG. 22. LOAD-DEFLECTION DIAGRAMS FOR TEST FLOOR OF SHREDDED WHEAT FACTORY.

13. *Deflections.*—Little use has been made in this report of the deflections which were measured in the tests, but since in many cities the building regulations make certain requirements for deflection under load, it is believed that the presentation of deflection data may serve a useful purpose. In Fig. 22 are given diagrams in which the ordinates represent the load in pounds per square foot, and the abscissas represent the deflection in inches. Accompanying these diagrams are the numbers of the deflection points, the locations of which are shown in Fig. 6.

14. *Summary of Results.*—The principal results brought out in the foregoing discussion are as follows:

1 There was a considerable increase in the deformations in both steel and concrete during the fifty-six hours of retention of the load.

2 Upon the removal of the load from the six panels, there was an

increase in the deformations across the center line of the three panels which remained loaded (section of positive bending moment). There was a decrease in the deformations across the side edges of the area remaining loaded (sections of negative bending moment). There was also a decrease in the deformations in the direction of the side edge of the loaded area in those bars under the load which lay near this edge.

3 The positive bending moment for a panel width corresponding to the deformations measured in the reinforcing bars in one direction was found to average 0.021 Wl for a panel width; the negative bending moment found in the same way was 0.026 Wl. These values may be of interest in comparing the results of this test with the results of other tests. It must be understood, however, that these do not represent values of the bending moment coefficients which should be used in design.

4 The distribution of stresses in the reinforcement across panel edges and across panel center lines was substantially uniform, taking averages of the several sections. The variation from uniform distribution of the moment factors for these sections corresponds closely to variation in the slab thickness and in the distribution of the reinforcement.

5 The measurements show that a large bending moment was developed in the basement columns under a partial loading of the slab.

6 In the lintel beams cracks were found on the interior side near the ends which probably were caused by the twisting action produced by bending moment developed in the slab at its edge by the load on the wall panel.

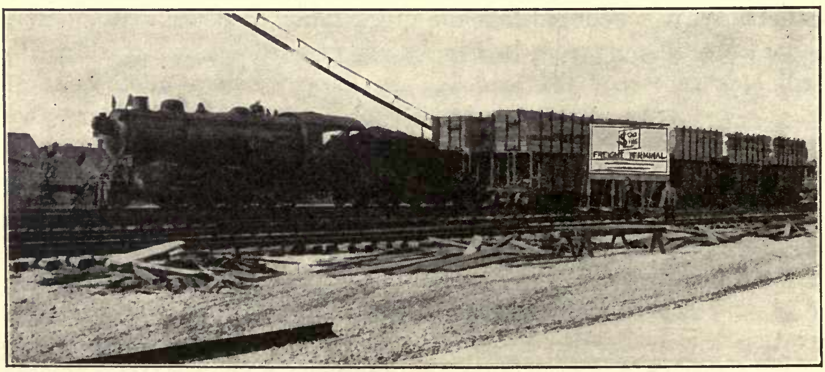


FIG. 23. VIEW OF TEST LOAD, SOO TERMINAL STRUCTURE.

III. The Test of the Flat Slab of the Soo Line Freight Terminal.

15. *The Structure.*—The Soo Line Freight Terminal of Chicago, comprises a freight yard and a central receiving and distributing building for the Central Terminal Railway Company of the Soo Line. The freight yard is elevated above the ground so that the intersecting streets cross underneath the terminal without a depression of their grades. Both across these streets and between them for a distance of more than half a mile the yard is on a reinforced concrete slab structure which is supported on columns. The space under the slab not occupied by streets is arranged to be utilized for storage purposes.

A more complete general description of the structure is given in Engineering Record, August 16, 1913 (Vol. 68, No. 7), in

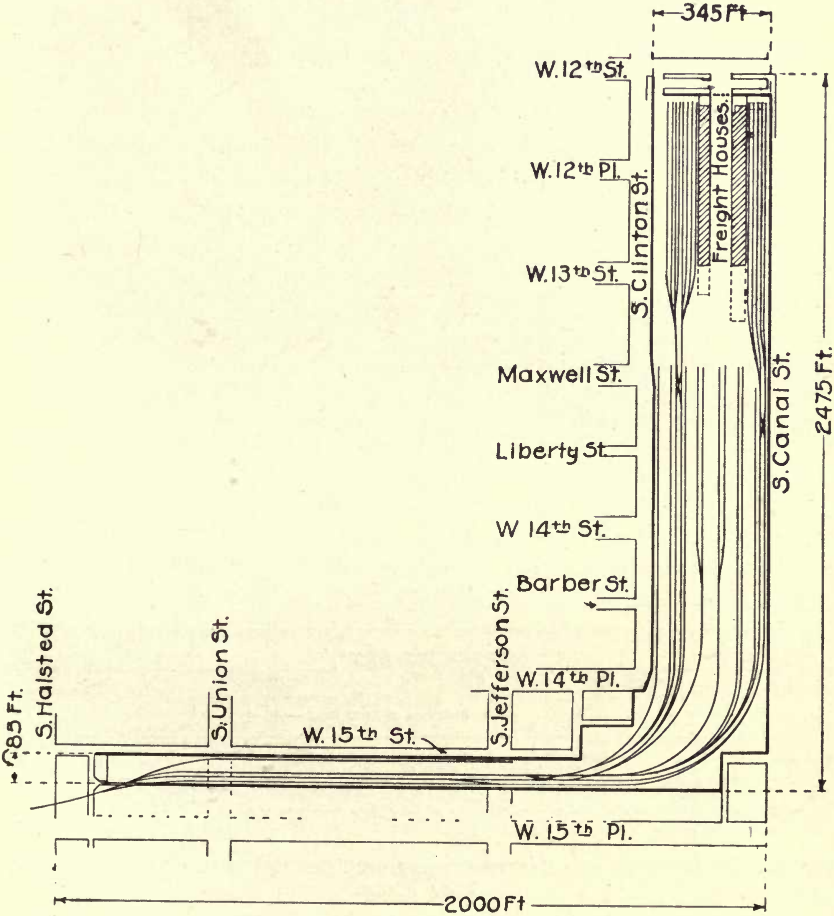


Fig. 24. General Plan of Soo Terminal Structure.

Engineering News, August 21, 1913 (Vol. 70, No. 8), and in Railway Age Gazette, August 22, 1913.

The general plan of the structure is shown in Fig. 24 and the design of a typical panel is shown in Fig. 25. For the test a four panel area was chosen which represents the typical flat slab of this structure, which is an unusually heavy one. The test area was located between West Fourteenth Street and Barber Street,

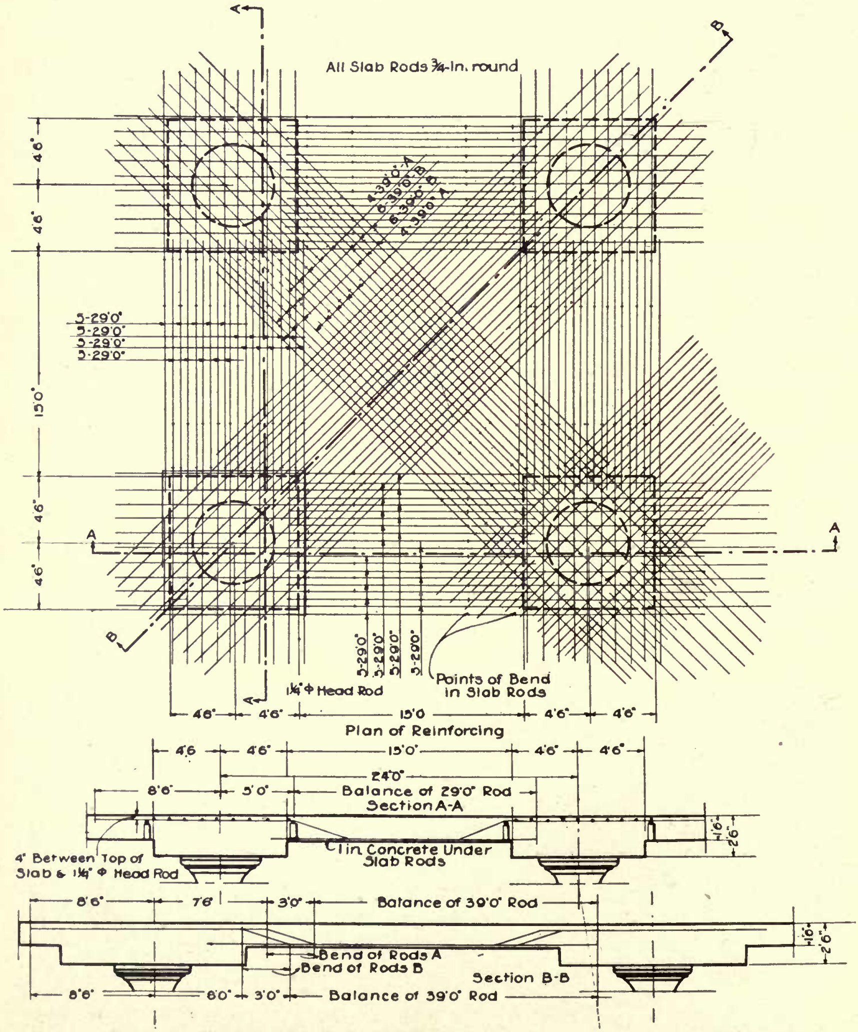


FIG. 25. DIMENSIONS AND REINFORCEMENT OF TYPICAL PANEL OF SOO TERMINAL TEST FLOOR.

Chicago. The test panels are 24 ft. square. The floor slab in the central portion of the panel is nominally 18 in. thick. Its nominal thickness is 30 in. throughout the area of a 9-ft. square surrounding each column, this portion of the slab being referred to as the depressed head. The necessary slope for drainage purposes was obtained by increasing the thickness of the slab to form alternate ridges and valleys. The thickness of the slab along the ridges was in some places as much as 2 in. greater than the nominal thickness. The additional thickness did not affect the position of the reinforcing bars. In regions of negative moment it is possible that an additional thickness of concrete affected the amount of the contribution of the tensile strength of the concrete to the resisting moment. In regions of positive moment an additional depth would increase the moment arm of the reinforcement stress.

The slab contains four-way reinforcement, as shown in Fig. 25. In slabs of ordinary thickness and span it has been a common practice to depend upon the weight of the reinforcing bars to pull the bars down into place in the center of the span, bars of one-half inch diameter or less being used. In a floor built for very heavy loading as in the case of the terminal slab under consideration, it is necessary to bend the bars to a definite shape. This bending was done after the steel had been placed in position. The average distance from the compression surface of the slab to the center of gravity of the reinforcement at the edge of the capital of column N63 was 26¼ in. The distance to the center of gravity of the reinforcement midway between columns varied somewhat due to the presence of the ridges previously referred to. The greatest and least depths found were 20¾ in. and 16¾ in., the least being fully as great as the depth which would be expected from the nominal thickness given above.

The columns are cylindrical, 32 in. in diameter, and end above in bell-shaped heads or capitals 5 ft. 6 in. in diameter at the top. The columns are reinforced with a spiral of ½-in. round steel, having a mean diameter of 28½ in. and a pitch of 2 in. In addition, 20 1⅛-in. diameter longitudinal reinforcing bars are placed just within the spiral and wired fast to it. The spiral is carried from a point 16 in. below the level of the basement floor to a point well into the bottom of the depressed head. Bearing and anchorage of the longitudinal rods are afforded by right angle hooks 8 in. long at the bottom and at the top. For the portion of the structure tested, the length of the columns from the top of the basement floor to the bottom of the depressed head averages about 12 ft. 10 in.

The footings are of reinforced concrete 14 ft. square with a concrete pier 4 ft. square extending from the top of the footing upward to a point 18 in. below the top of the basement floor.

16. *The Test.*—The concreting of the panels on which the test was made was done on June 12, 1913, the building being 112 days old at the time of the test, and the four-panel test load was applied on October 2, 1913. Ore cars having bins built on top to an average height of 6 ft. 2¼ in. above the regular height of the car were filled with broken stone and run upon the test area to furnish the load. Fig. 23 shows the loaded cars in position for the five-panel test (Load C).

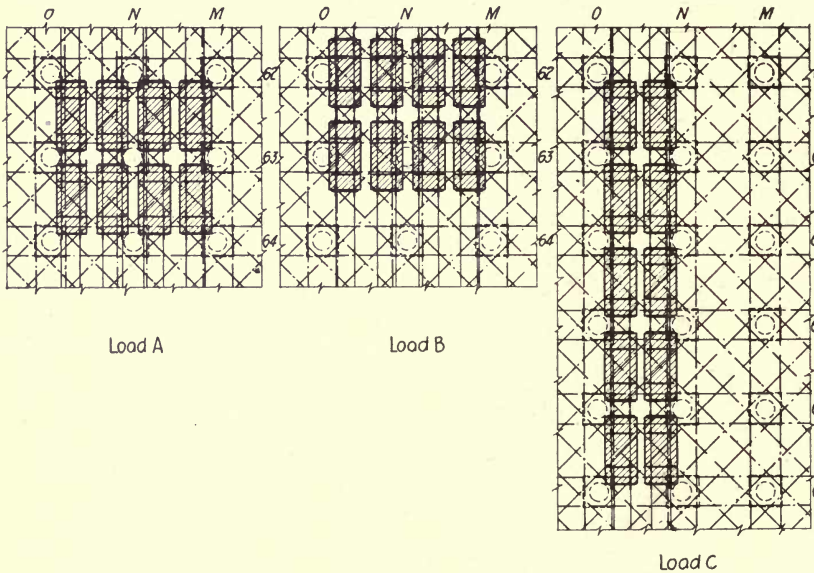


FIG. 26. PLAN SHOWING POSITIONS OF CARS FOR LOADS A, B, AND C ON SOO TERMINAL TEST FLOOR.

Portions of the ballasted and of the uncovered slab are seen in the foreground. The cars held on an average 1740 cu. ft. each., and the average weight including the car was 200,800 lb. The intensity of this load is considerably greater than the Terminal Railway Company expects the slab to be subjected to in service. Table 3 gives data of the loaded cars. It may be said in anticipation of results reported hereinafter that this load caused stresses in the slab so small that their interpretation is difficult.

The three different positions of load, designated as A, B, and C, are indicated in Fig. 26. Load A was designed to give the representative stresses around a central column approaching the condition of all panels loaded. The movement into position B was designed to determine the effect of a change of a half panel length in the position of the load. Load C was designed to find the effect of loading adjacent tracks lying within a row of panels.

The preparation for the test covered the time from Sept. 17 to Sept. 29, inclusive. As showing the extent of the preparations, it may be stated that during this time from eight to twelve laborers were employed in cutting or drilling into the concrete to expose the steel

TABLE 3.

WEIGHTS OF LOADING MATERIAL AND DIMENSIONS OF CARS.

Average weight of cars alone	32,000 lb.
Average weight of stone in cars normally	74,600 lb.
Average additional height of bin	6 ft. 2¼ in.
Average length of bin inside	18 ft. 7½ in.
Average width of bin inside	8 ft. 5¼ in.
Average additional capacity	970 cu. ft.
Average unit weight of stone used	97 lb. per cu. ft.
Average additional weight of stone per car	94,200 lb.
Average total weight of car and stone	200,800 lb.
Number of cars loaded	10
Maximum variation of total weight of any car from average	400 lb.
Length of car center to center of coupling	24 ft.
Center of coupling to center of first wheel	2 ft.
Center of first wheel to center of second wheel	5 ft.

and to place gage lines for measurements on the concrete. The test occupied the time from Sept. 30, 1913 to Oct. 8, 1913. Observations were taken on 816 gage lines, of which 338 were on the upper surface and 478 on the lower surface of the slab. Locations of slab gage lines are shown in Fig. 27, and 28 and locations of deflection points in Fig. 30. Locations of gage lines on columns may be determined by a study of Fig. 37 to 41 inclusive. In the carrying out of the test considerable delay was experienced because of rains which flooded the gage holes and necessitated a large amount of tedious work in draining and cleaning.

In order to lessen the chances of error in readings, two complete sets of strain gage readings were taken before applying a load, and also two complete sets under Load A (see Fig. 26). The load was then moved to the position indicated as load B, Fig. 26, and two series of readings were taken in only the significant positions. The cars were then removed from the test area and a complete series of no-load readings was taken. With Load C in position readings were taken at the most important positions.

17. *Settlement and Deflections.*—Levels were taken on the slab at several of the columns and at deflection points before loading,

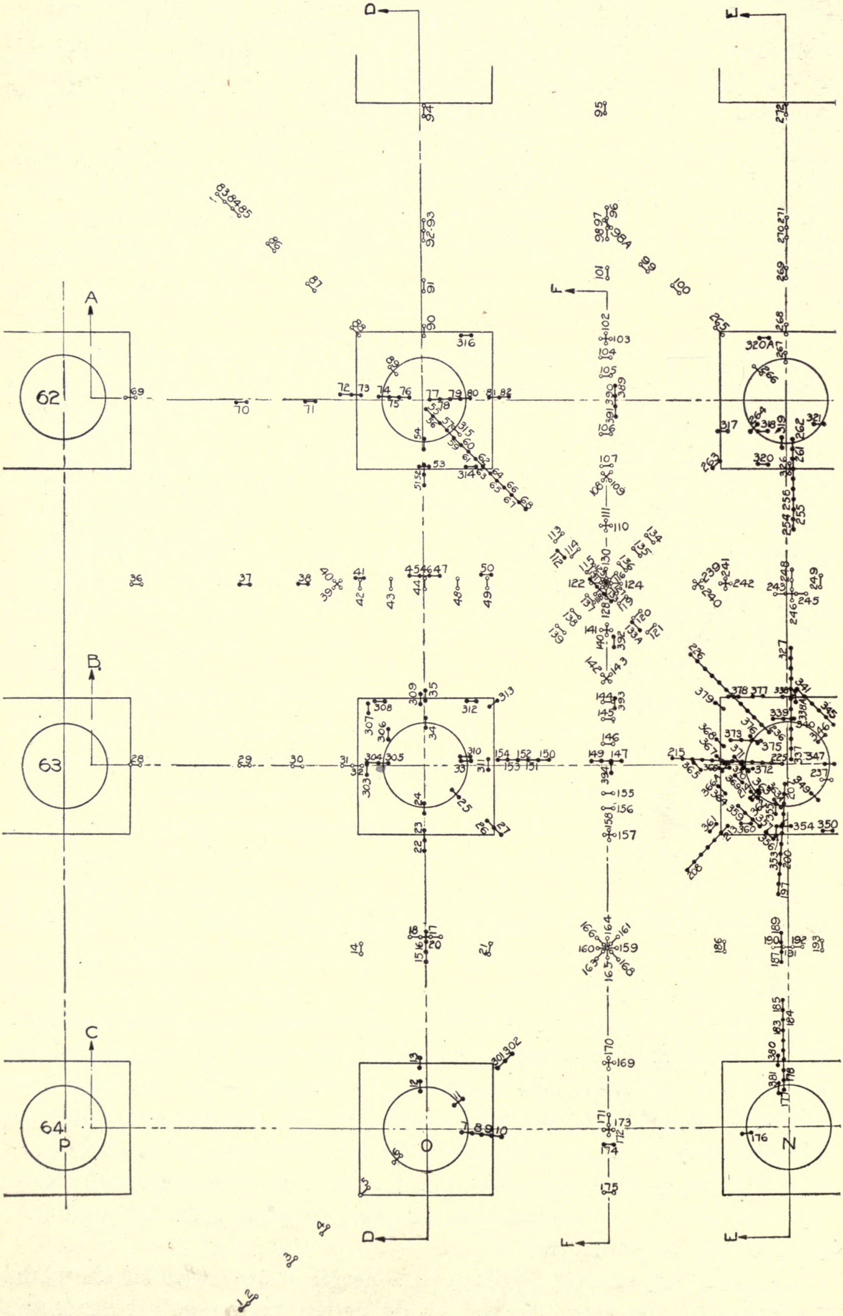
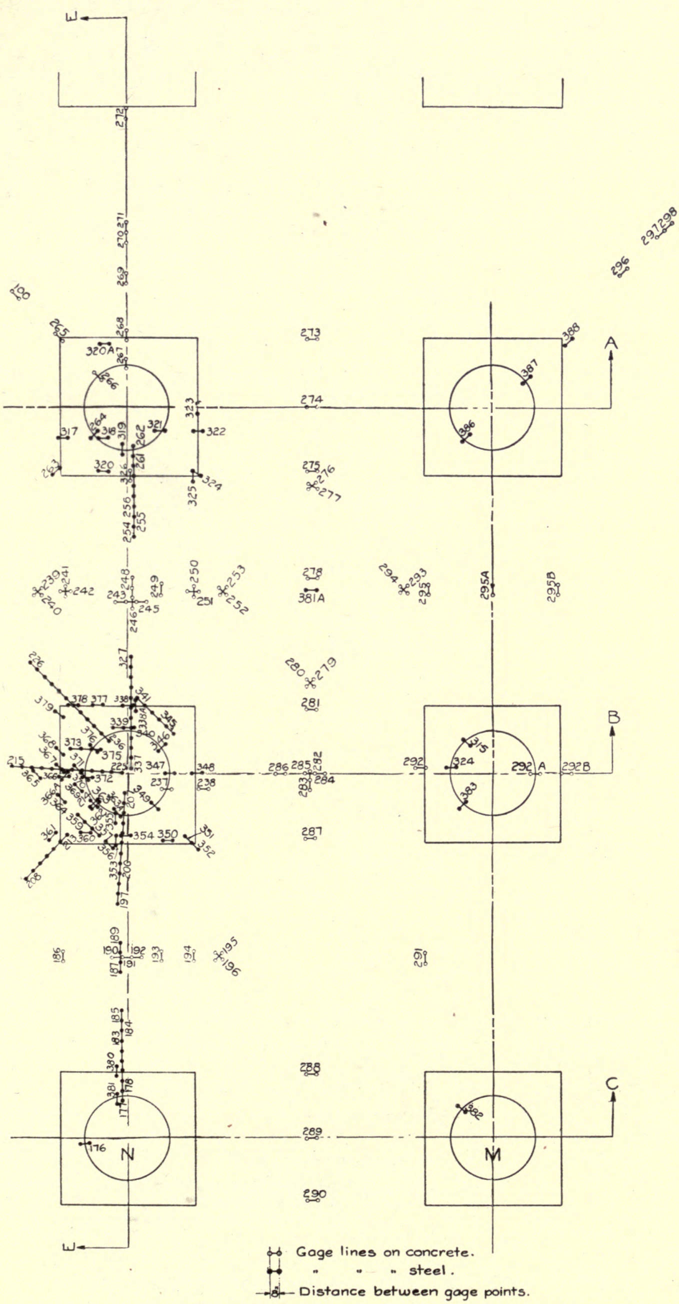


FIG. 27. LOCATION OF GAGE LINES



ON UPPER SURFACE OF SOO TERMINAL TEST FLOOR.

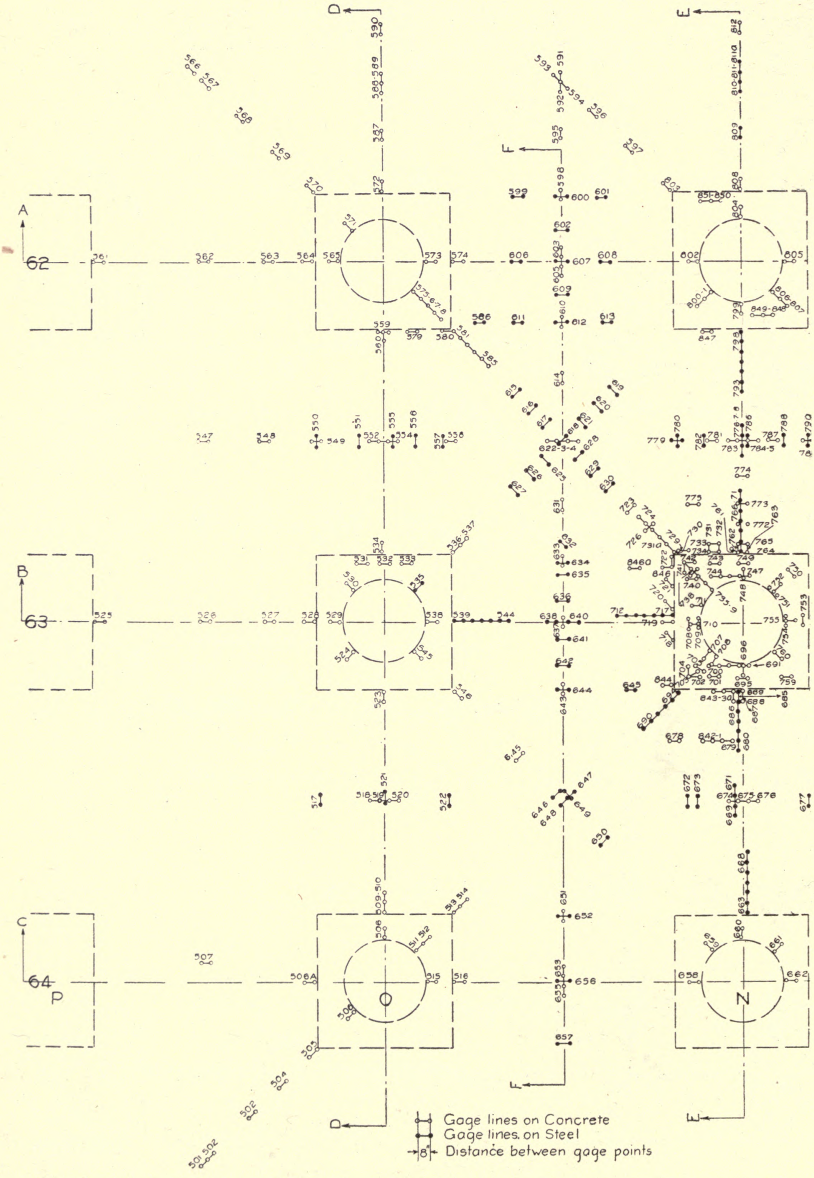
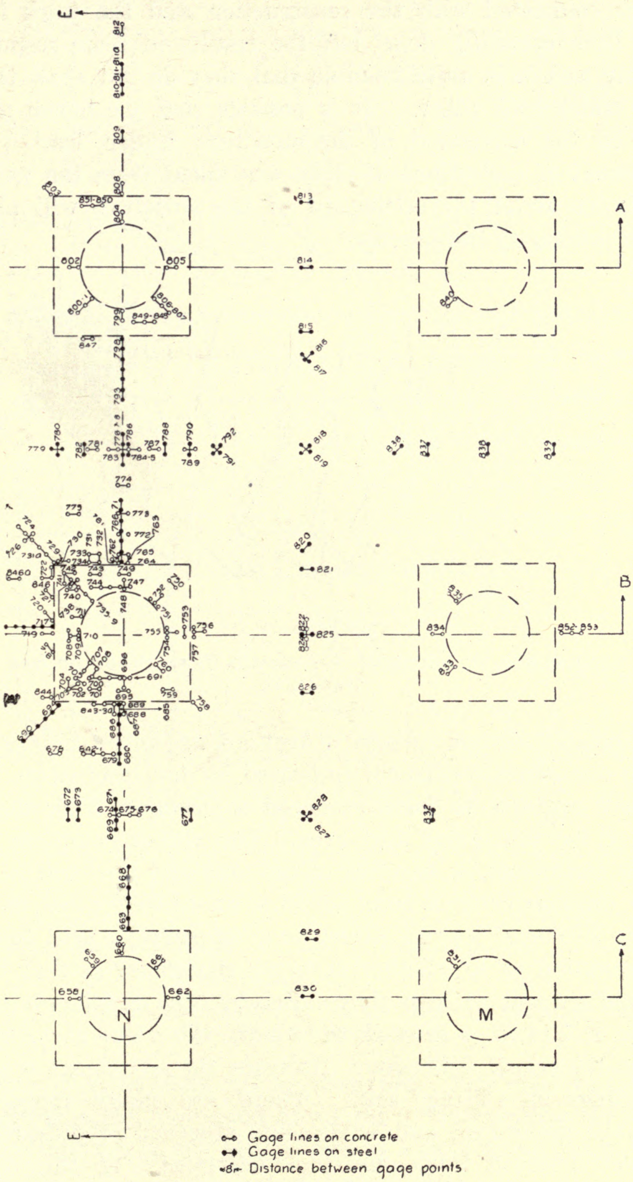


FIG. 28. LOCATION OF GAGE LINES



ON UNDER SURFACE OF SOO TERMINAL TEST FLOOR.

with load on, and with load removed. The levels were taken by engineers connected with the construction and the work is known to have been carefully done, but the results in some respects seem conflicting and it is quite possible that they do not show the actual changes which took place. It is possible that the bench mark was affected by the movement of the structure under load. As the measurement of deflections of slabs was taken from the floor below, uncertainties about the settlement of the structure will also affect the observations on deflections.

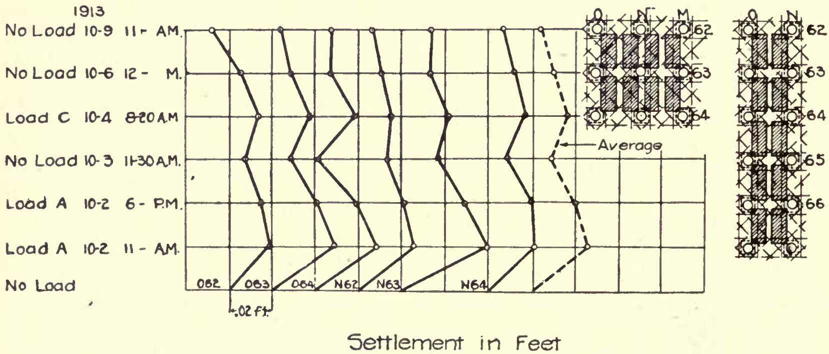


FIG. 29. SETTLEMENT OF COLUMNS SUPPORTING TEST FLOOR OF SOO TERMINAL STRUCTURE.

The data of settlement and deflections are given in Fig. 29 and 30. The maximum settlement indicated by the level notes was 0.04 feet which appears to have occurred at Column N63, the central column of the four-panel test area. The almost complete recovery from this settlement on removal of the load and the large amount of recovery during the time from 11:00 a. m. to 6:00 p. m., October 2, while the full load was in place, seem improbable and hence raise serious question as to whether as much settlement as that indicated above was present at any time. Besides, the maximum measured deflection of 0.14 in. is so small as to make this value also seem not to represent the actual deflection. However the settlement is not large for buildings on Chicago soil. There are many inconsistencies between the deflections and settlements observed at different places and under different loads, and the uncertainty of the conditions makes it futile to attempt to account for the recovery from settlement or otherwise to interpret the data. However, Fig. 29 and 30 are given for the purpose of record. It is evident that even slight settlement under load will modify the distribution and

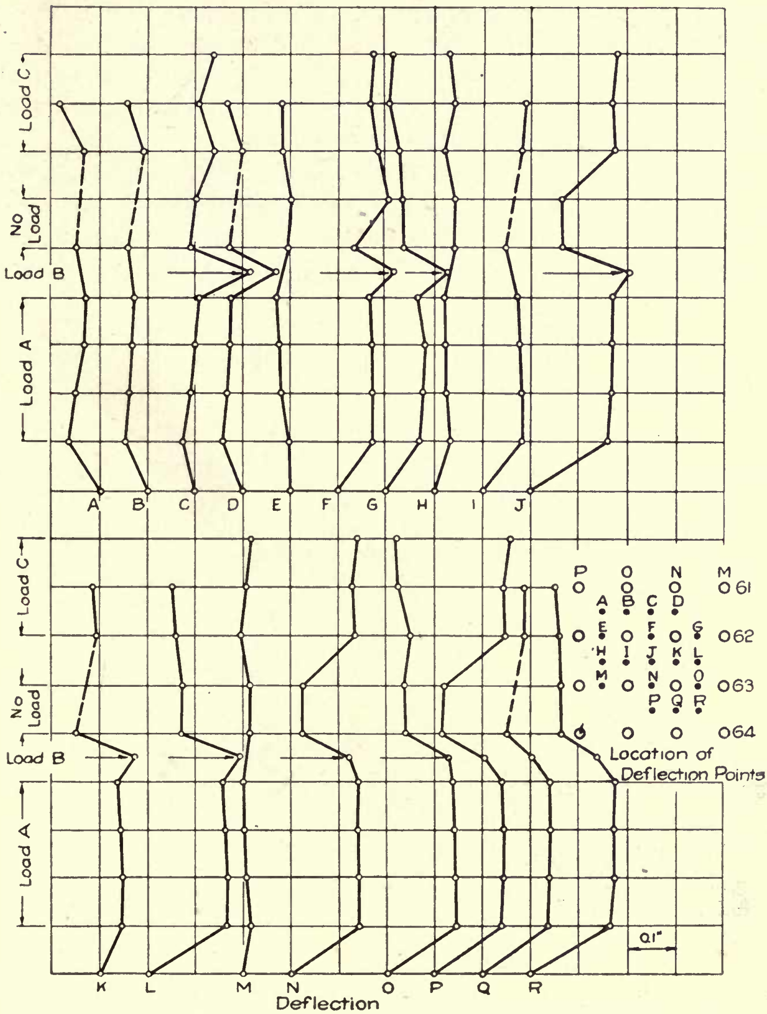


FIG. 30. DEFLECTION OF TEST FLOOR AT POINTS MIDWAY BETWEEN COLUMNS OF SOO TERMINAL STRUCTURE.

the amount of the stresses in the structure. The reference to settlement is given in the discussion as an aid in interpreting the observed phenomena and not in any way as having a bearing on the stability of the structure.

18. *Deformation in the Slab.*—The diagrams in Fig. 31 to 34 give the results of measurements taken to determine the distribution of deformations in the slab along the edges of panels, the measurements being taken in the direction of these edges, that is, along or parallel

to the sections shown in these figures. The diagrams in Fig. 35 and 36 give the results of measurements taken to determine the distribution of deformations across the center line of panels (section F-F) on the top and on the bottom of the slab. In this case the measurements were taken normal to the sections shown.

The deformations measured in the slab for the several loads were very small, in general, much smaller than were anticipated. As a result the data are not such as to throw much light on several of the questions on which information was sought, as for example, the effect

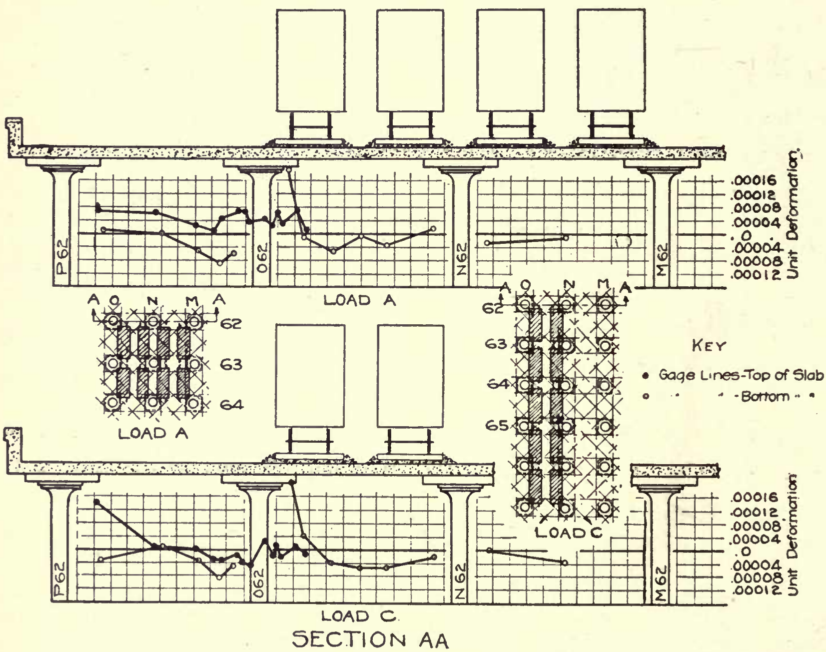


FIG. 31. DEFORMATION ALONG SECTION A-A (NORMAL TO TRACKS) OF SOO TERMINAL TEST FLOOR.

of shifting the load a short distance. In many cases the changes in length were smaller than the possible errors of observation, and such results are therefore meaningless. Besides, with the low stresses found, the tensile strength of the concrete must have played an important but uncertain part in the resistances developed. To complicate the matter further, unequal settlement of the footings evidently modified the action of the structure. Altogether, it may be said that on account of the smallness of the deformations and complications with the tensile resistance of concrete and the settlement of the foot-

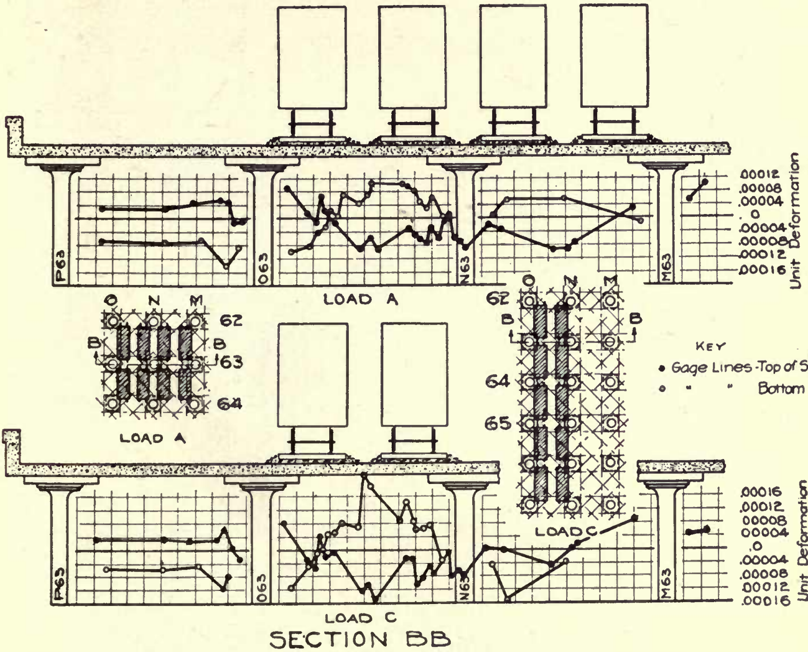


FIG. 32. DEFORMATION ALONG SECTION B-B (NORMAL TO TRACKS) OF SOO TERMINAL TEST FLOOR.

ings little conclusive information was obtained on the main questions connected with the action of the slab, except that the smallness of the stresses developed may be considered as important in judging of the action of such flat slab structures. In some respects the observations give indications of the way in which the stresses are distributed, but frequently they are so masked by uncertainties that comparison cannot be made with any degree of confidence. However, comment will be made as best it can.

Some of the difficulties may be seen from the following examples. For the negative bending moment around the central column (column N63, Fig. 32), with four panels loaded, the compressive stresses in the concrete due to both dead and live loads would not be expected, from calculations made according to current practice, to exceed 500 lb. per sq. in. If the concrete on the tension side remained intact, the tensile stresses in the concrete in this region would be less than this amount and the stresses in the reinforcement would be correspondingly low. As no cracks were observed in these positions and as the observed tensile deformations here were very small, it seems probable that the tensile strength of the concrete was sufficient

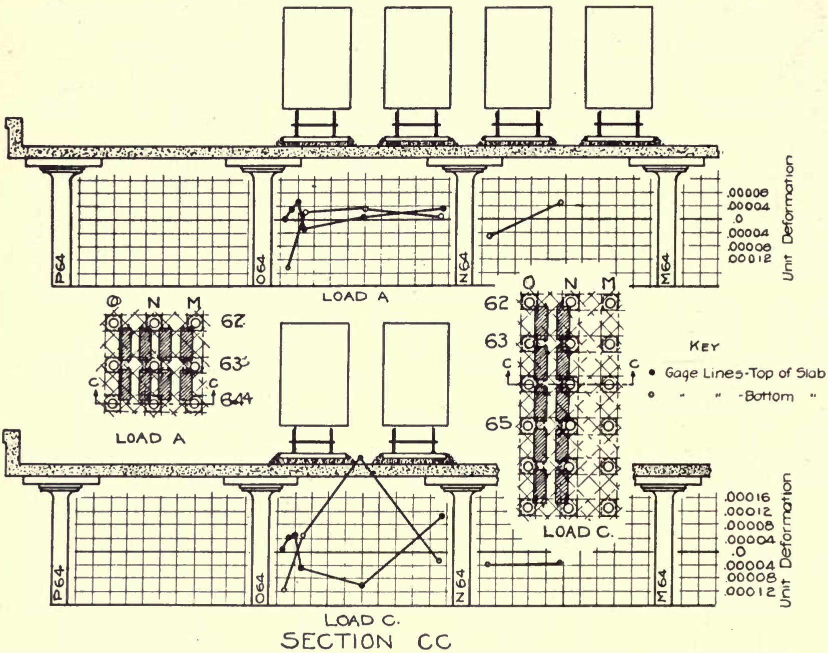


FIG. 33. DEFORMATION ALONG SECTION C-C (NORMAL TO TRACKS) OF SOO TERMINAL TEST FLOOR.

to prevent cracking in regions of negative moment even without relief of the stresses here by settlement of the column. The differences in depth of embedment of the reinforcing bars in the different bands also added to the uncertainties. In view of these difficulties, no quantitative values of the negative bending moment developed in the slab around the central column can be given. Again, for the four-panel loading, the highest deformation measured in any of the reinforcing bars of the bands at the central column corresponded to 2700 lbs. per sq. in. tension in the bar and to, say, 270 lb. per sq. in. tension in the concrete on the upper surface of the slab and the range was from this value to a small compressive deformation. It is evident that with such stresses as these it would be of little value to try to make comparison of stresses in the different bands or in different bars of the same band.

The deformations in the reinforcing bars of the rectangular bands at the bottom of the slab half way from the central column to the adjoining column were larger than those in the top of the slab near the central column (see Fig. 32). The settlement of columns may have had some influence on this. Even here the highest deformation

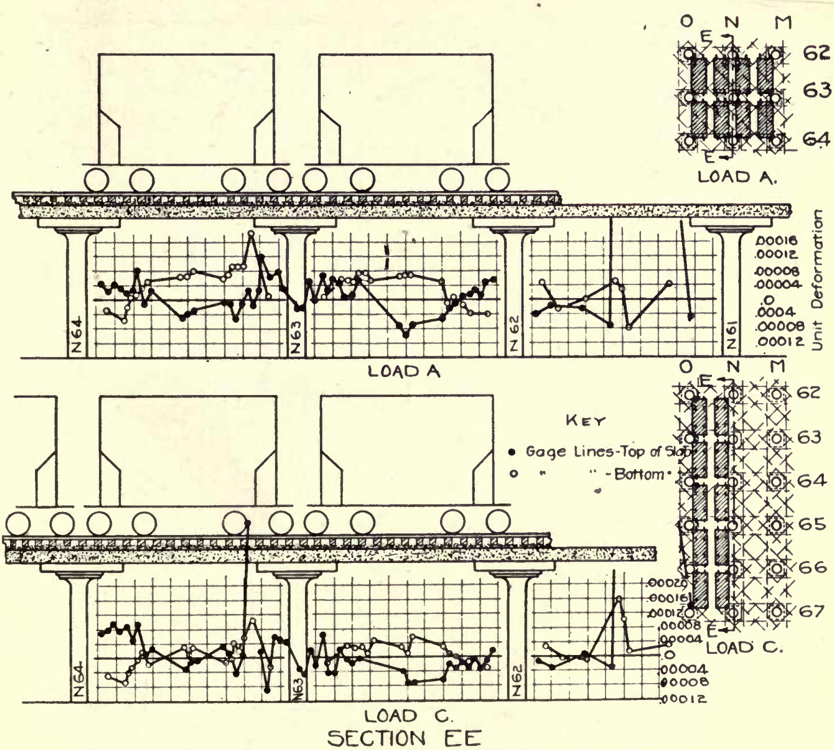


FIG. 34. DEFORMATION ALONG SECTION E-E (PARALLEL TO TRACKS) OF SOO TERMINAL TEST FLOOR.

measured in any bar under load A corresponds to a stress of less than 4000 lb. per sq. in., and most of the bars show values much smaller than this.

In the four-panel loading (see plan of load A, Fig. 26) the bars in a band of reinforcement which has load on both sides of it (as the rectangular band from Column N62 to N63 and on to N64, and also the rectangular band from Column M63 to N63 and on to O63) may be expected to take their full share of the stresses due to the load. In bands at the edge of the loaded area (as in the band extending from Column O62 to O63 to O64) the bars at the edge of the band which lie within the loaded area will be stressed higher than the bars at the other edge of the same band which lie outside the loaded area, but the latter bars will carry a considerable proportion of stress, and the stresses developed in the former bars will be materially less than if the latter bars were not present. This condition will need to be taken

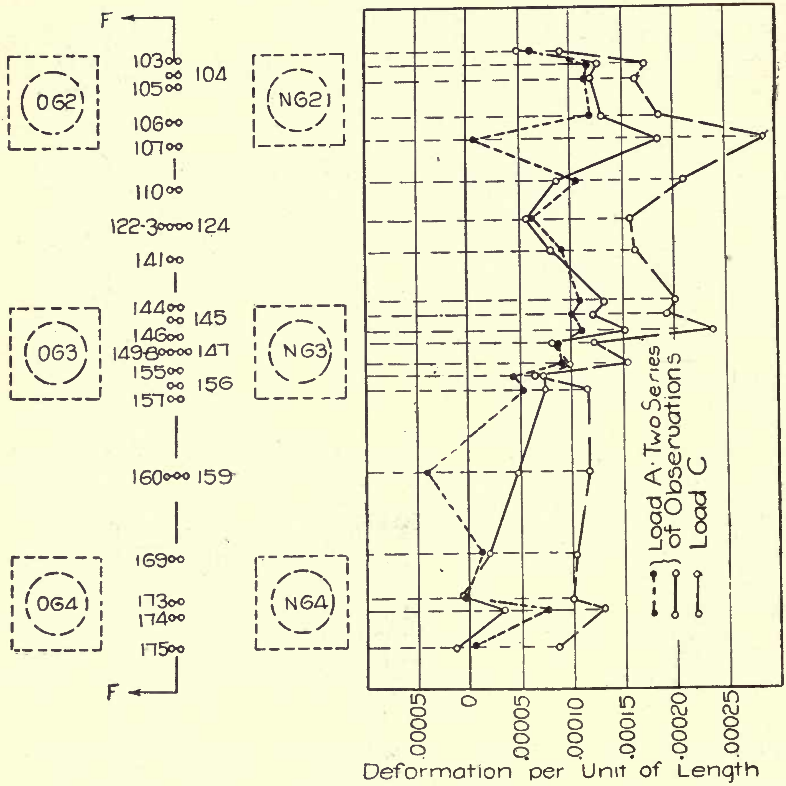


FIG. 35. EFFECT OF CHANGE FROM LOAD A TO LOAD C ON DEFORMATION IN GAGE LINES CROSSING SECTION F-F ON TOP OF SOO TERMINAL TEST FLOOR.

into consideration in comparing the deformations found when the load is removed from a panel but is left on the adjoining panel.

In the change from a four-panel loading (load A) to a load on a row of panels (load C), besides the condition just referred to, a change may be expected in the stresses in the bands which run across the row of loaded panels. Due to the bending of any column under the new arrangement of load (like Column N63, Fig. 32, which in the four-panel loading was surrounded by a symmetrical load) and with the resulting change in slope at the top of the column, and due also to whatever reverse bending occurs in the slab outside the loaded area, it may be expected that for a band such as the one running from Column N63 to O63 the stresses at Column N63 (due to negative bending moment) would be less for the row of panels loaded than it

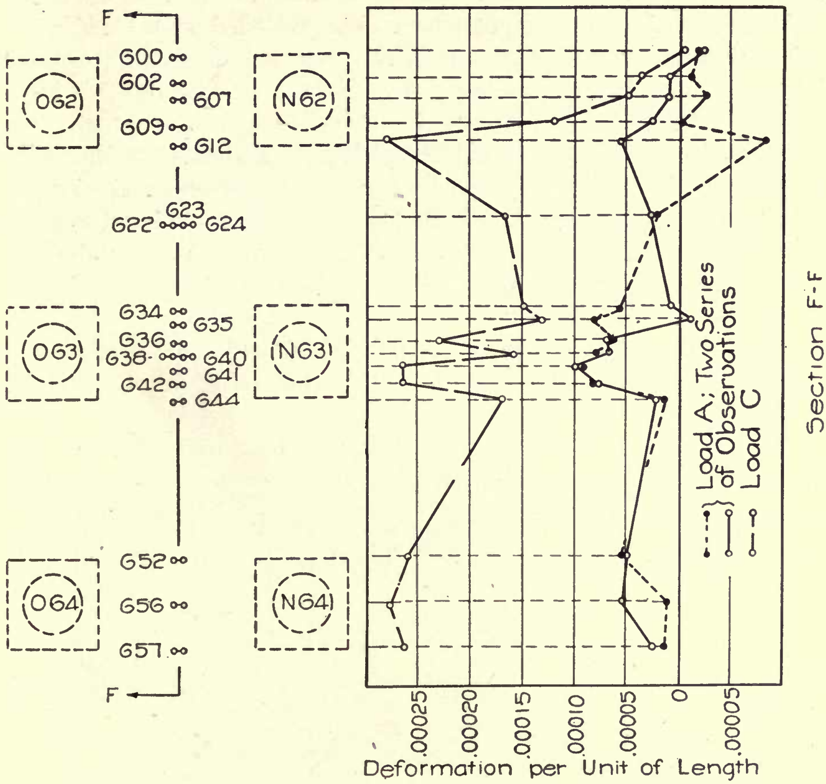


FIG. 36. EFFECT OF CHANGE FROM LOAD A TO LOAD C ON DEFORMATION IN GAGE LINES CROSSING SECTION F-F ON BOTTOM OF SOO TERMINAL TEST FLOOR.

had been for the four-panel load, and that at points half way between Columns N63 and O63 the stresses (due to positive bending moment) would show an increase over those for the four-panel load. Section C-C, Fig. 33, shows the effect of loading a row of panels and of extending the load entirely across a section instead of loading on only one side of the section. This effect is seen in the considerable increase in deformation midway between Columns N64 and O64 (see also Fig. 35 and 36.)

When the loading was changed from load A to load C the measurements at Column N63 showed very little change in the stresses in the bars of the band which extends from Column N63 to O63 (see Fig. 32). At a point half way between these columns the stresses in the bars increased, averaging about 1500 lb. per sq. in. with load A and 6000 lb. per sq. in. with load C (see Fig. 32 and 36.) Taking into account the stresses produced by the dead load, it seems that the tensile resistance of the concrete must have been exceeded with load C, and this makes impracticable the use of the resulting stresses for comparing quantitatively the bending moments developed by the two loads.

For the band running from Column O62 to O63 the average stress observed in the reinforcement at mid-span under the four-panel loading (load A) was only 500 lb. per sq. in. and under the loading of a row of panels (load C) 900 lb. per sq. in. For the band running from Column N62 to N63 (see Fig. 34), the average stress observed at mid-span under load A was 2000 lb. per sq. in. and under load C 500 lb. per sq. in.

It will be seen that for load A, there is considerable similarity in the distribution of stresses along section B-B (Fig. 32) and section E-E (Fig. 34).

19. *Columns.*—Columns located at the edges of the loaded area showed considerable bending stress. The stresses in columns were

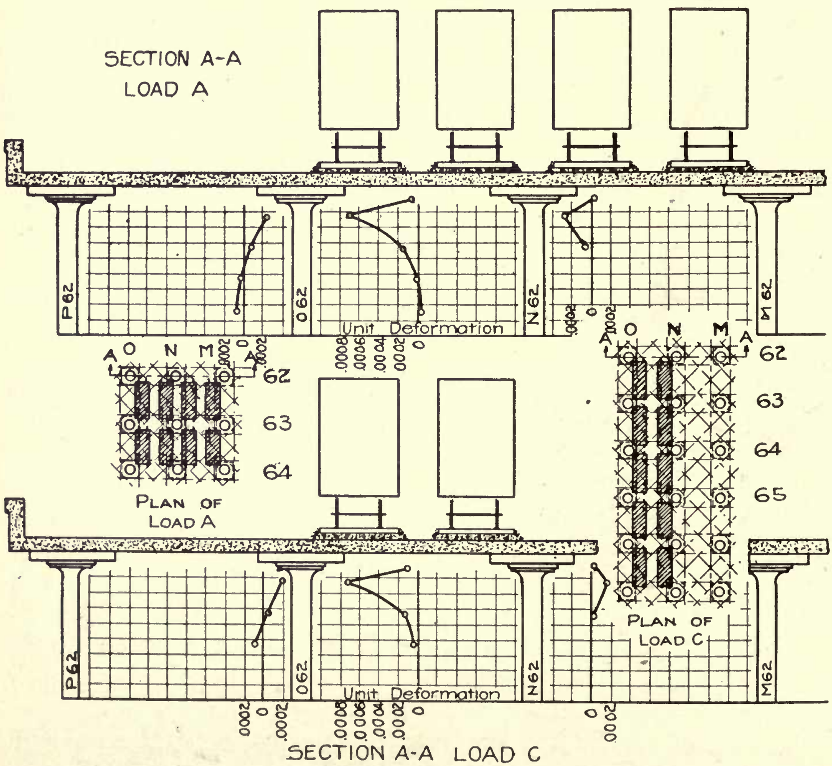


FIG. 37. DEFORMATION IN COLUMNS CUT BY SECTION A-A (NORMAL TO TRACKS) OF SOO TERMINAL STRUCTURE.

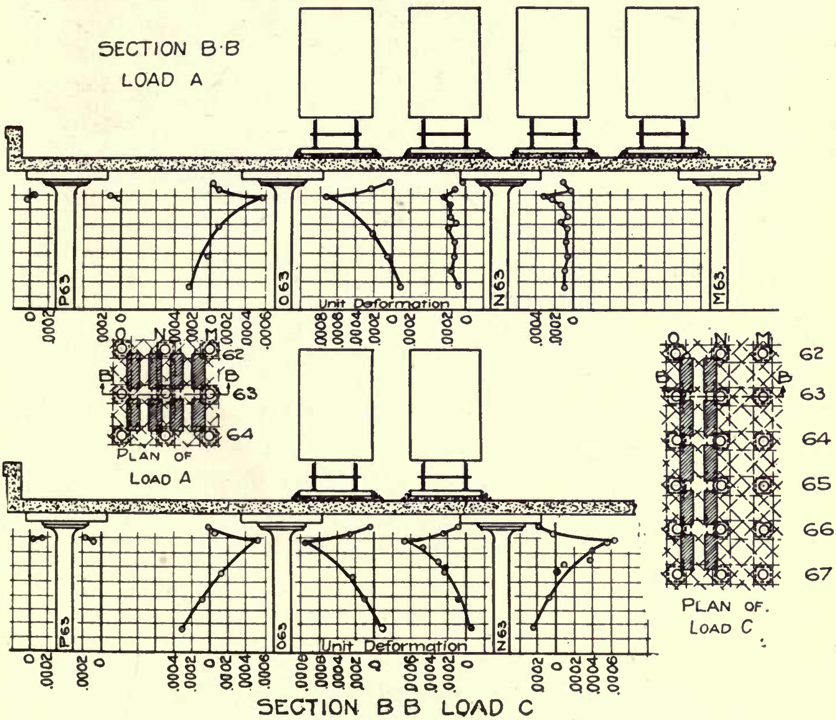


FIG. 38. DEFORMATION IN COLUMNS CUT BY SECTION B-B (NORMAL TO TRACKS) OF SOO TERMINAL STRUCTURE.

higher and more definite than the stresses in the slab. The amount of the deformations in the concrete on opposite sides of the column is shown graphically in Fig. 37 to 41. The large tensile deformations in the columns show that the reinforcing bars were subjected to considerable tensile stress, and even considering the compression due to dead load the tensile strength of the concrete must have been exceeded. This action is also shown by the formation of cracks on the tension side of the columns. Measurements made on the reinforcing bars (not plotted in the figures) show tensile deformations smaller than those observed in the concrete, but still considerable. The distribution of the flexural deformations along the column length is also shown in Fig. 37 to 41. In general the point of zero deformation is lower on the compression side than on the tension side of the column, a difference which may be due in part to the direct compression caused by the test load. Table 4 gives the distances from the bottom of the depressed head to the point of zero deformation on the two sides of their columns and their averages. The distances are

Table 4.

Positions of Points of Zero Deformation on Columns.

Column No.	Load	Distance in Inches from Depressed Head to Point of Zero Deformation			Ratio of Average to Column Length
		Loaded side	Unloaded side	Average	
063	A	East 115	West 81	98	.64
063	C	East 105	West 85	95	.62
N63	C	West 106	East 78	92	.60
062	A	East 103	West 103	103	.67
062	C	East 98	West 78	88	.57
064	A	East 108	West 106	107	.70
064	C	East 108	West 100	104	.68
064	A	North 106			
062	A	South 103			
062	C	South 105			

expressed also as proportional parts of the total distance between the bottom of the depressed head and the upper surface of the basement floor. The average of the ratios is 0.64. If the case of a column

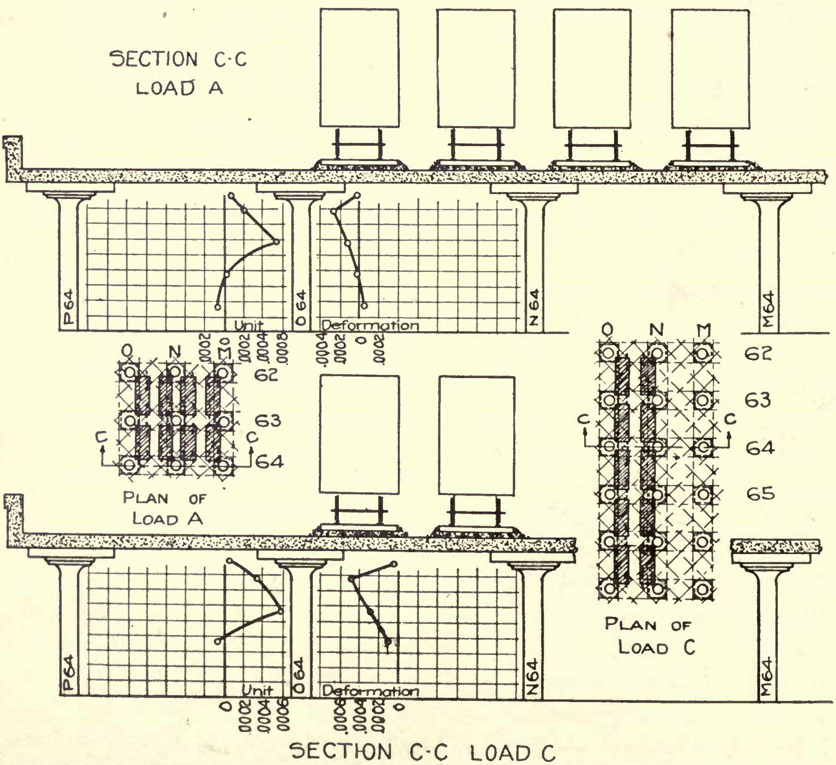


FIG. 39. DEFORMATION IN COLUMNS CUT BY SECTION C-C (NORMAL TO TRACKS) OF SOO TERMINAL STRUCTURE.

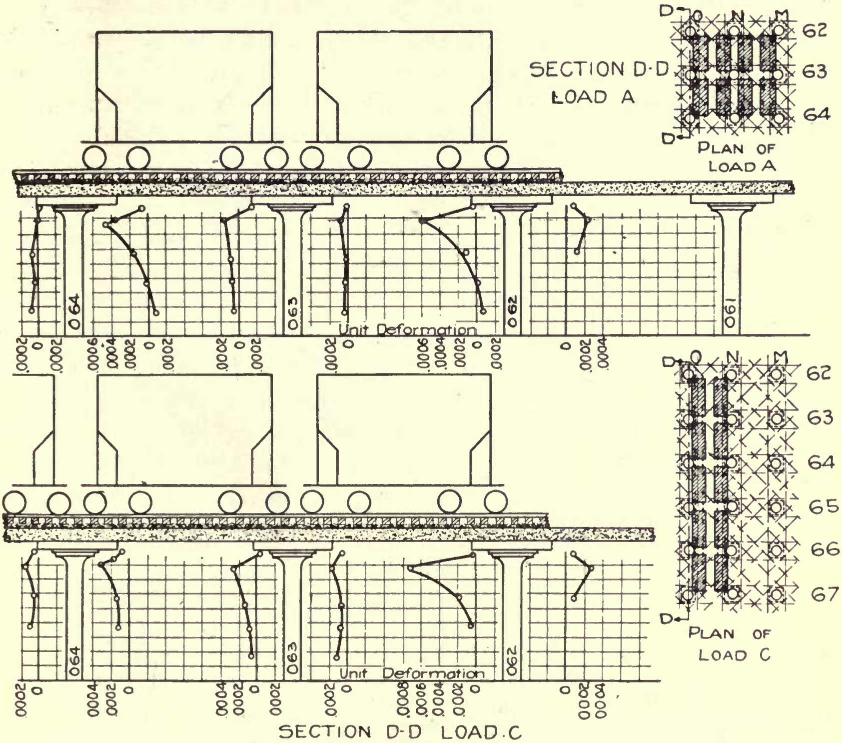


Fig. 40. Deformation in Columns Cut by Section D-D (Parallel to Tracks) of Soo Terminal Structure.

fixed at one end and having a rigid connection with the slab or beam at the other end be subjected to analysis, the point of inflection will be found to be two-thirds of the distance from the point of rigid connection to the point of fixity. The results are in fair agreement with the analysis if the column be considered to be fixed at the basement floor.

Because of the uncertainty of the effect of tensile stresses in the concrete and because the value of the modulus of elasticity of the concrete of the columns is unknown, it is not feasible to calculate the bending moment produced in the columns. It can easily be seen, however, that the bending moment developed in this case is very large, as may be expected from an analysis of a structure made up of thick slabs, and not having the columns continued upward to other stories. The fact that such bending is developed in columns and is shown by measurements may be worth recording.

20. *Cracks.*—The location of small cracks which were observed

in the slab when the four-panel load (load A) was in position is shown in Fig. 42. All these cracks were found on the bottom of the slab. No cracks were found on the top. It is seen that in general they extend in a direction parallel with the rows of columns, displaying also a slight tendency to extend along the diagonal of the panel. Portions of the upper side of the slab were covered with ballast and no examination for cracks could be made in these places, but as the ballasted regions occupied the central portions of the panels where compression should be found cracks would not be expected at these places. The top of the slab was available for examination over all columns, and no cracks could be found in such positions although a careful search for them was made. It is seen that the cracks referred to are in regions where compression would be expected. Their appearance may be explained by even a slight settlement of columns; those on the under side of the slab close to the depressed heads of some of the columns at the edge of

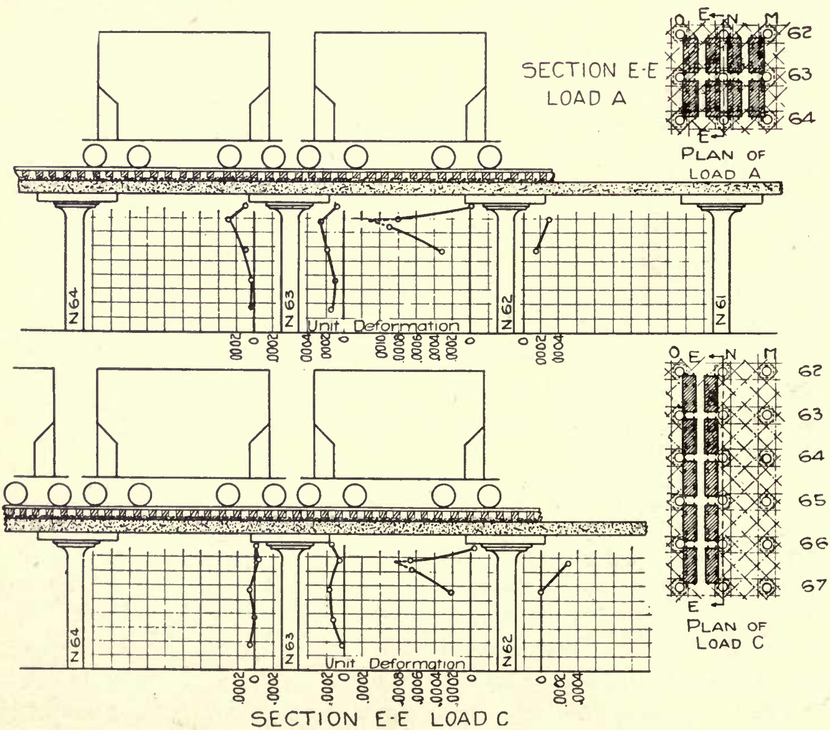


FIG. 41. DEFORMATION IN COLUMNS CUT BY SECTION E-E (PARALLEL TO TRACKS) OF SOO TERMINAL STRUCTURE.

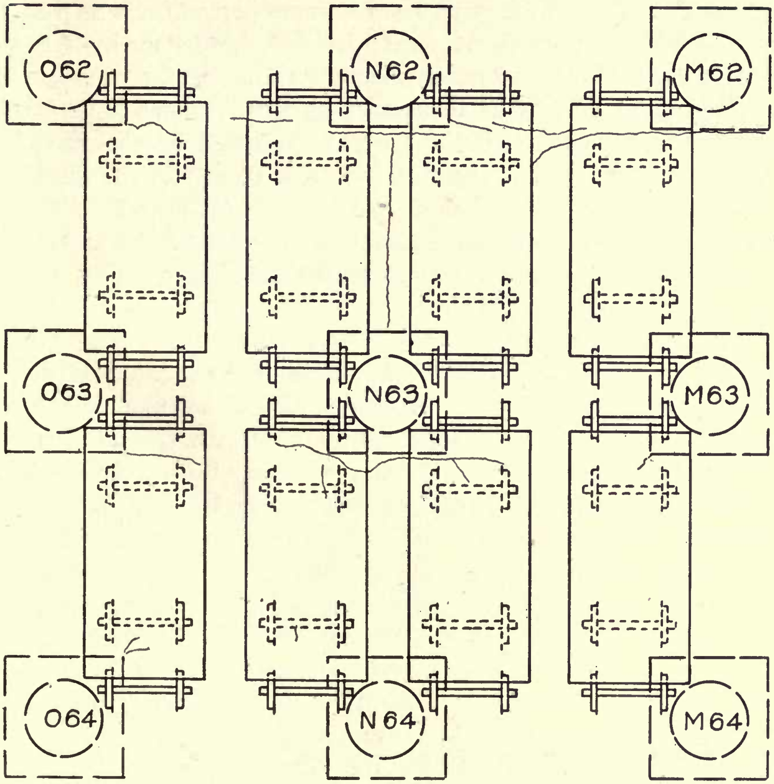


FIG. 42. LOCATION OF CRACKS IN SOO TERMINAL TEST FLOOR.

the loaded area may be explained if there were settlement of these columns, and those which were found on the under side of the slab close to the depressed head of the central column if there were a settlement of this column larger than that of any other.

Under Load A, cracks were observed in several columns located on the edges of the loaded area. These were found on the column capital a little above its bottom edge and on the side away from the loaded area. It was not easy in every case to distinguish between these cracks and other cracks found on the columns which evidently were due to expansion and contraction of the floor caused by changes in temperature, but a survey of the temperature cracks disclosed that they were systematically arranged and could be distinguished from the cracks which were due to eccentricity of loading and which were indicative of bending moment developed in the columns as already discussed under 19. *Columns.*

21. *Measurement of Dead Load Stress.*—An effort was made to obtain information on the amount of stress developed in the slab by the weight of the floor upon the removal of the forms. The test was made upon a 24 by 24-ft. panel. As the floor slab is very thick it cannot be expected that the stresses developed by the dead load would be very high, but the test indicates that it is practicable to measure the effect produced when the forms are removed.

The forms were left standing until the time of the test. Gage lines were placed where readings were desired and initial or zero readings were taken. The forms were then removed and other sets of readings were taken when the slab was supporting its own weight. In order to place gage lines on the bottom of the slab before the forms were removed, it was necessary to cut holes in the forms. As the deformations were expected to be small, special care was taken to insure correctness of the observations. Both initial readings and final readings on all gage lines were taken twice by each of two observers.

TABLE 5.

AVERAGE UNIT-DEFORMATION UNDER DEAD LOAD.

Plus indicates extension and minus indicates shortening.

Location	Average Observed Unit-deformation
Bottom; outside of capital; rectangular direction	—.000042
Top; outside of capital; rectangular direction	+.000013
Bottom; outside of capital; diagonal direction	—.000024
Top; outside of capital; diagonal direction	+.000005
Bottom; outside of capital; Depressed head, rectangular direction	—.000013
Top; outside of capital; Depressed head, rectangular direction	+.000005
Bottom; midway between columns; rectangular direction	+.000013
Top; midway between columns; rectangular direction	—.000048
Bottom; midway between columns; diagonal direction	+.000014
Top; midway between columns; diagonal direction	—.000032

The averages of unit deformations found are given in Table 5. It is seen that for measurements at gage lines located near the column capital the deformations were greater on the bottom of the slab than on the top, while for the locations midway between columns, the deformation was greater on the top of the slab than on the bottom; in other words, the deformation was greater in the regions of compression than in the regions of tension. From the amounts of the deformations measured it may not be expected that the concrete had failed in tension. In judging of the amount of these deformations it must be considered that there is a possibility that some of the dead weight is assumed by the slab in advance of the removal of the forms because of the shrinkage of the forms or of very slight settlement. The fact that the compressive deformations are higher than the tensile deformations may indicate that arch action played a part in the support of the dead load.

22. *Summary of Results.*—The following comments may be made on the test of the Soo Terminal Building:

1 The deformations measured in the steel and in the concrete of the slab were very small, in many cases smaller than the possible errors of observation. The tensile stresses developed in the reinforcement being so small, the tensile strength of the concrete must have played a very large part in the bending resistance of the slab. It appears also that uneven settlement of the footings under the applied load modified the action of the structure.

2 With the development of such low stresses and the uncertain action due to uneven settlement of the footings, the results of the test may not be used to throw light on the mechanics of the slab and on the distribution of stresses over the slab in the way it was hoped they could be used. As would be expected, an increase in the stress in a cross band under the loaded area was found when the load was changed from four panels in the form of a square to five panels in a row.

3 Marked bending was found in the columns at the edge of the loaded area. The point of inflection of the elastic curve of flexure of the columns was about two-thirds of the distance from the bottom of the depressed head to the upper surface of the basement floor, which is the location to be expected for a column fixed at the bottom and having a rigid connection with the slab at the top.

4 The location of the cracks found on the under side of the slab indicates that stresses in a structure subject even to slightly uneven settlement of footings may be of different character from those found by the ordinary assumptions of design.

5 The measurements made to determine the stresses produced in the slab by dead load upon striking centers indicate that while the deformations were small in the test made, it is practicable to measure the deformations in a reinforced concrete structure due to dead load.

IV. The Schulze Baking Company Building Test.

23. *The Building.*—The building of the Schulze Baking Company is located at 55th St. and Wabash Ave., Chicago. It is five stories

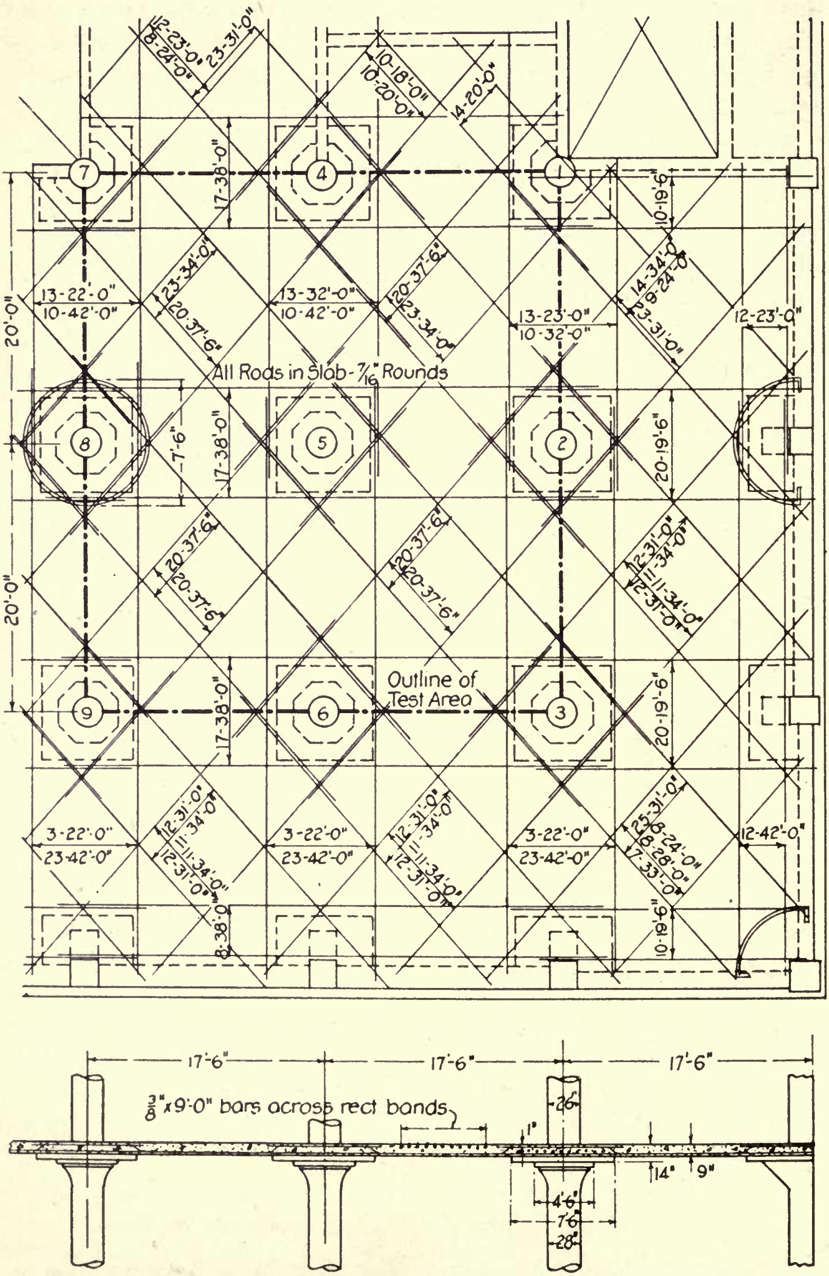


FIG. 43. DIMENSIONS AND REINFORCEMENT PLANS OF TEST FLOOR OF SCHULZE BAKING COMPANY BUILDING.

in height and covers an area 298 ft. 4 in. by 160 ft. The floor space is divided into panels 17 ft. 6 in. by 20 ft. The test load was applied to panels of the second floor (one story above sidewalk level). The floor construction in this part of the building consists of a four-way reinforced concrete slab nominally 9 in. thick in the central portion of the panel and 14 in. thick throughout the area of a 7-ft. 6-in. square surrounding each column. It was designed for a live load of 300 lb. per sq. ft.

The columns below the second floor are circular, 28 in. in diameter, and terminate at the top in octagonal bell-shaped heads

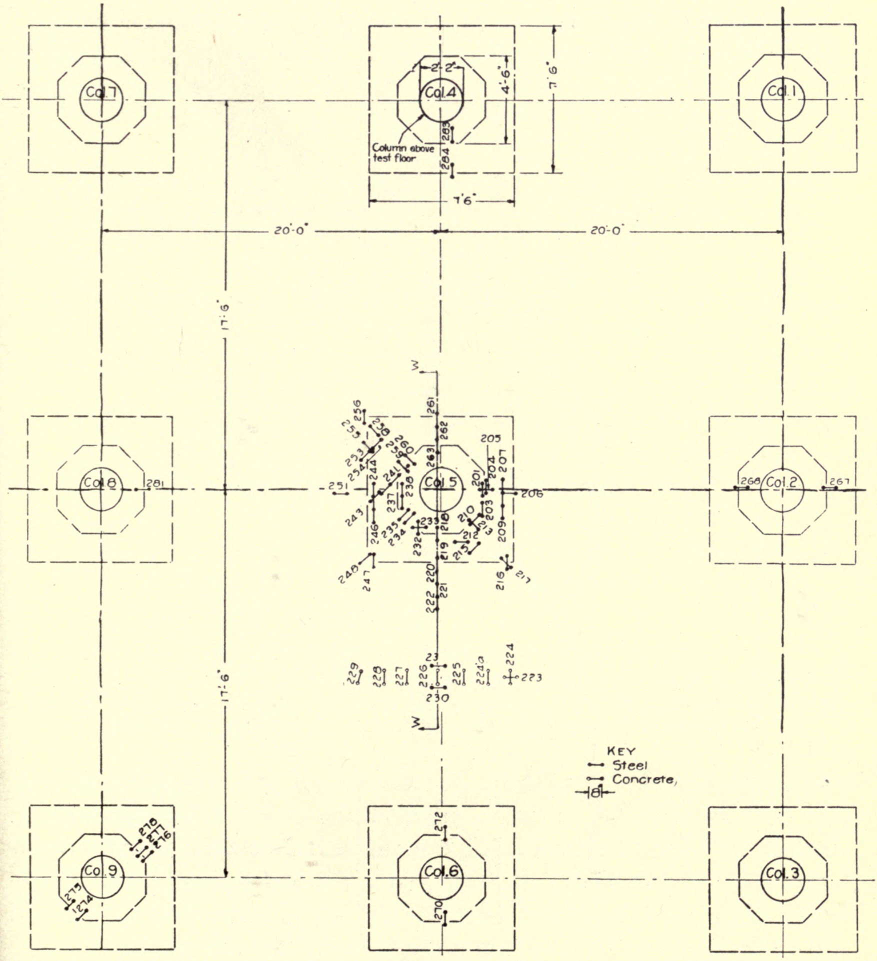


FIG. 44. GAGE LINES ON UPPER SURFACE OF TEST FLOOR OF SCHULZE BAKING COMPANY BUILDING.

54 in. in diameter at their upper ends. The columns between the second and third floors are circular, and are 26 in. in diameter.

The distribution of the reinforcement in the portion of the floor to which the test load was applied is shown in Fig. 43.

Two-thirds of the slab rods of the rectangular bands are located in the top of the slab where they pass over the columns. These rods drop down to the bottom of the slab at a point about two-tenths of the panel length from the center line of the columns. At the corresponding point relative to the next column the rods are bent up again and pass over the column in the top of the slab, ending 9 in.

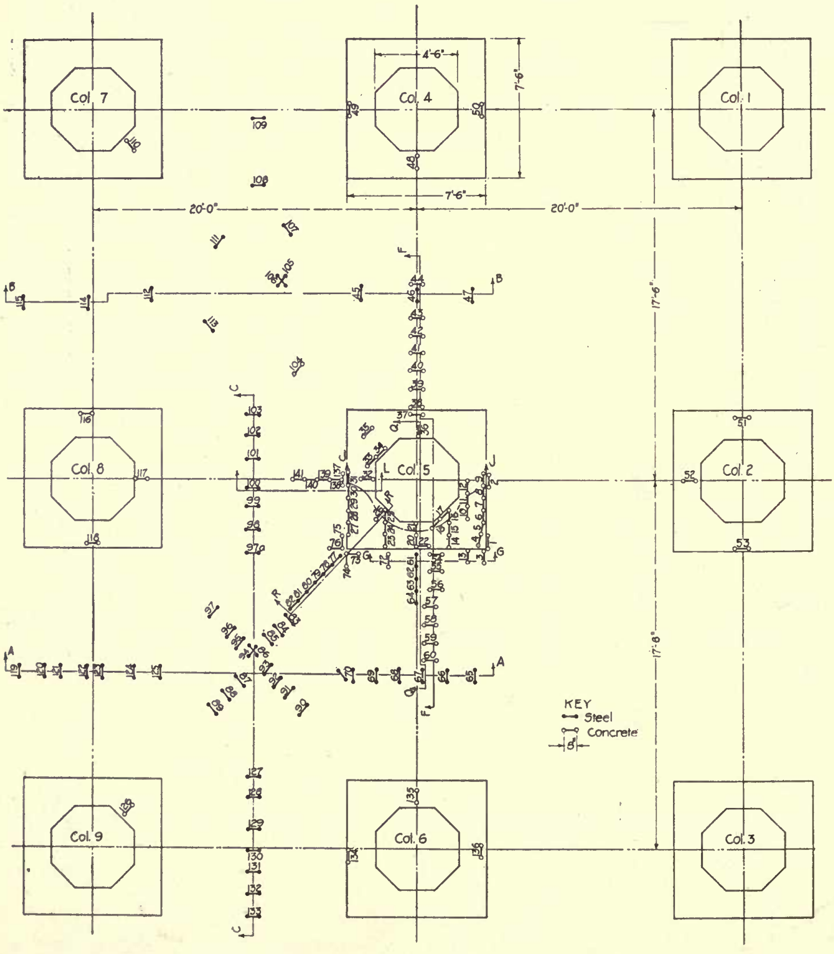


FIG. 45. GAGE LINES ON UNDER SURFACE OF TEST FLOOR OF SCHULZE BAKING COMPANY BUILDING.

to 18 in. beyond the center line of columns. The remaining one-third of the bars of the rectangular bands extend through the bottom of the slab throughout the length of the bars. All diagonal rods bend up to pass over the column in the top of the slab, and extend 5 ft. 3 in. beyond the center lines of columns. Measurements taken at the time of the test indicate an average depth from the compression surface to the center of gravity of the reinforcement of 10.15 in. for positions of negative moment and 7.45 in. for positions of positive moment.

To prevent concentration of cracks along the center line of the band of reinforcement from column to column, 3/8-in. bars 9 ft. long spaced about 12 in. apart were placed across the boundary line between panels.

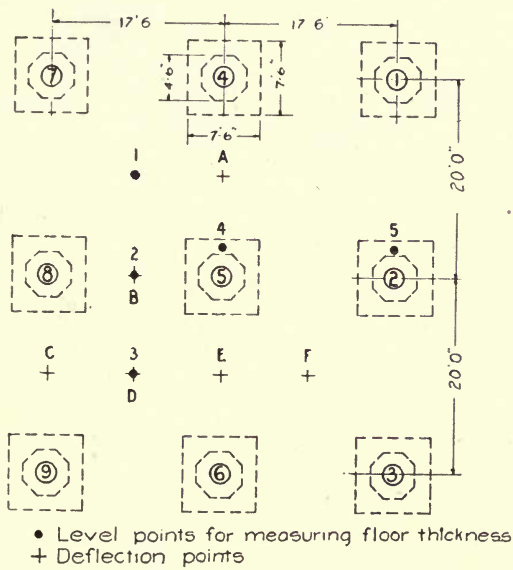


FIG. 46. LOCATION OF POINTS FOR MEASUREMENT OF THICKNESS AND OF DEFLECTION OF TEST FLOOR IN SCHULZE BAKING COMPANY BUILDING.

The concrete was placed late in October, 1913, and there was considerable cold weather during the time of hardening. For this reason it was expected that the concrete would not show up as well as concrete poured and set under the more favorable conditions of summer, and because of the unfavorable conditions a little more time than usual was allowed before making the test. At the time of the test it appeared that there still was considerable moisture in the concrete, but no unfavorable indications were found in the concrete.

24. *The Test.*—The period of preparation for the test covered the time from January 13 to January 19, 1914. Deformations were measured in 123 gage lines on the reinforcement of the slab, 82 gage lines on the concrete of the slab, and 58 gage lines on the concrete of the columns. The gage lines on the upper and under surfaces of the slab are shown in Fig. 44 and 45. All the strain gage readings were taken by Mr. Slater. Deflections and floor thicknesses were measured in five places, the locations of which are shown in Fig. 46. The floor thicknesses for the various points in order were as follows: (1), 9⅛ in.; (2), 8 3/16 in.; (3), 8 11/16 in.; (4), 14⅛ in.; (5), 14 7/16 in. This gives an average thickness for the thin portion of the floor of 8.87 in. and for the thick portion of 14.28 in.

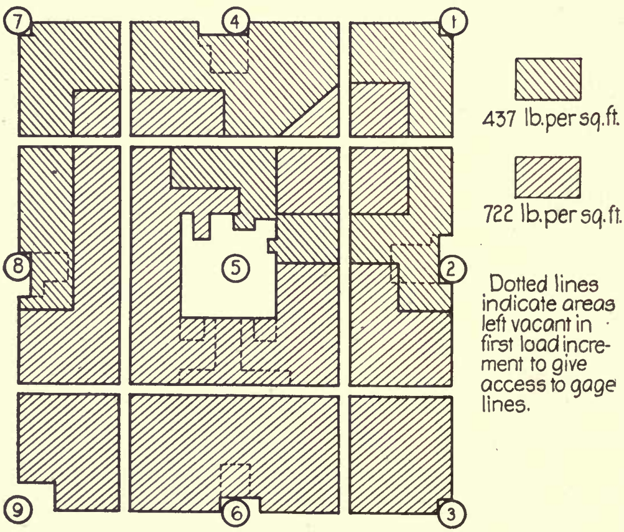


Fig. 47. Plan Showing Distribution of Load on Test Floor of Schulze Baking Company Building.

Brick was used as the loading material. Previous to the test its weight was determined by weighing a number of wheelbarrow loads, noting both the number of bricks and measuring the cubic contents when stacked. The weight per cubic foot was found to be 96 lb. The load was applied to four panels as shown in Fig. 47, the brick being stacked in piers. Fig. 48 is a view of the load in place. On account of shortages of brick the load was not always uniformly distributed over the floor at the time of taking the readings. Table 6 shows the loads at which strain gage readings were taken and indi-

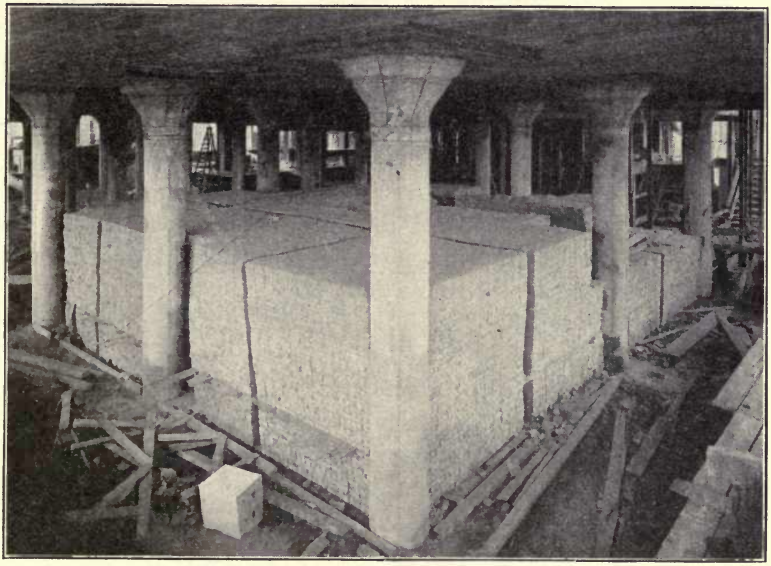


FIG. 48. VIEW OF TEST LOAD ON FLOOR OF SCHULZE BAKING COMPANY BUILDING.

cates the nominal loads used in plotting the load-deformation curves.

TABLE 6.

LOADING OF FLOOR IN SCHULZE BAKING COMPANY BUILDING.

Stage of Loading	Height in inches	Courses of Brick	Load lb. per sq. ft.
1	29	12	189
2	48¼	20	319
3	65	27	437
4	106	44	722

The maximum load was 722 lb. per sq. ft. (twice the design live load plus the dead load), except for a portion of the two north panels (Fig. 47) which had 437 lb. per sq. ft. The deficiency was due to a shortage of material, but the load on these panels was distributed with a view to making the material at hand as effective as possible in producing bending moment. It is believed that the moment in the fully loaded panels, both at the central column and midway between columns was not less than 90 per cent of the moment which would have been developed if the load had been 722 lb. per sq. ft. over the entire area.

Expansion and contraction of the steel and concrete with change in temperature made complications which partially obscure the results of the test. In an effort to eliminate possible errors from this source, corrections were determined by taking readings on an unstressed gage line in the floor. In spite of the indications that the expansion and contraction of the concrete go through the same cycles of changes as the temperature of the air, corrections made on the basis of these observations do not remove all the inconsistencies from the load-deformation curves. However, it is believed that such errors have been greatly reduced by the corrections used.

25. *Tension in Slab.*—The measured unit-deformations observed in this test were unusually small, both in the steel and in the concrete. Very few unit-deformations were more than 0.0002 and the majority were less than this at the maximum load of 722 lb. per sq. ft. Assuming a modulus of elasticity of 30,000,000 lb. per sq. in. for steel, the stress at a unit deformation of 0.0002 is 6,000 lb. per sq. in.

TABLE 7.

AVERAGE STRESSES IN TENSION REINFORCEMENT AT MAXIMUM LOAD.

Gage Lines	Location and Direction of Gage Lines	Average Stress lb. per sq. in.
203, 208, 216, 232, 237	Long rectangular direction at central column	7200
205, 212, 233	Short rectangular direction at central column	3900
215, 210, 234, 235, 242, 253	Diagonal across loaded area at central column	5400
272, 283	Long rectangular direction at columns at edge of loaded area	4300
268, 281	Short rectangular direction at columns at edge of loaded area	3300
276, 277, 278	Diagonal at column at corner of loaded area	7100
65 to 70, inclusive	Long rectangular direction midway between columns	8700
97a to 103, inclusive	Short rectangular direction midway between columns	6300
90 to 97, inclusive	Diagonal across loaded area midway between columns	3900
83 to 89, inclusive	Diagonal across corner of loaded area midway between columns	3000
127 to 133, inclusive	Long rectangular direction midway between columns on edge of loaded area	4900
119 to 125, inclusive	Short rectangular direction midway between columns on edge of loaded area	3600
267	Short rectangular direction just outside of column at edge of loaded area	1200
274, 275	Diagonal just outside of column at corner of loaded area	2800

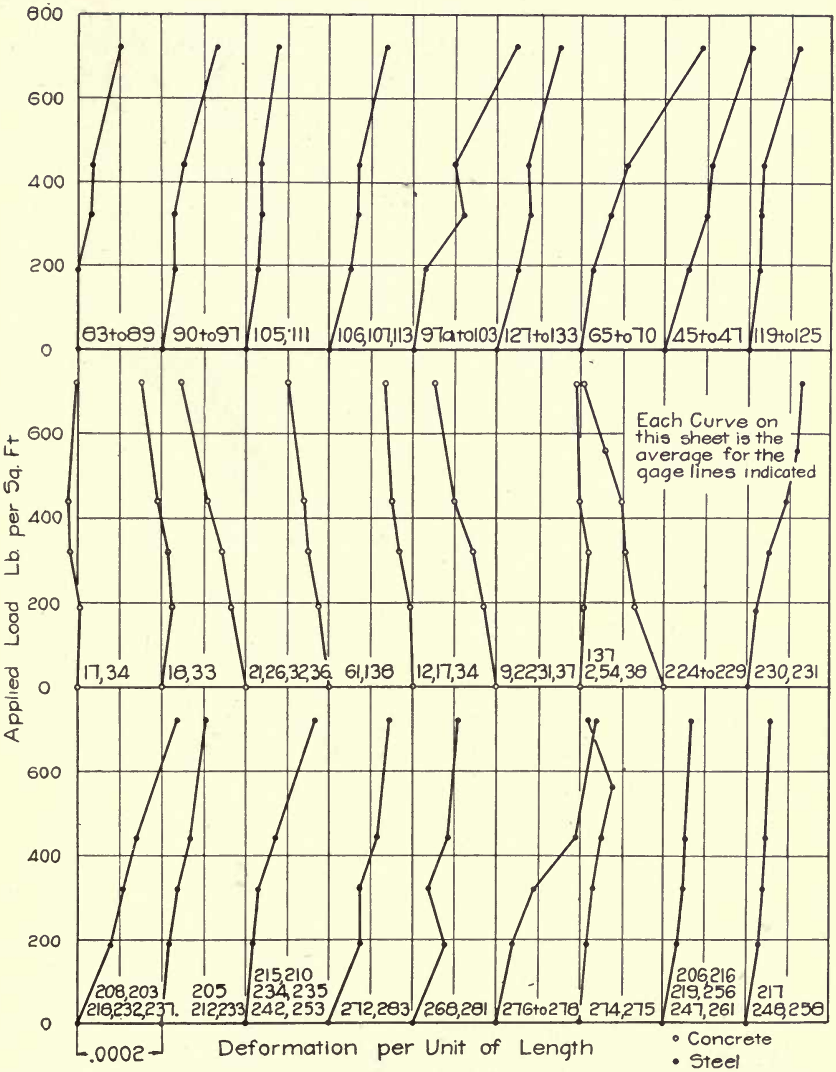


FIG. 49. AVERAGE LOAD-DEFORMATION DIAGRAM FOR TEST IN SCHULZE BAKING COMPANY BUILDING.

There were only two cases in which a value as high as 10,000 lb per sq. in. was reached.

In Table 7 the average stresses in steel for groups of gage lines at various locations are shown. While these figures may indicate correctly the relation of the stresses of the several groups, it is recognized that in a test where stresses generally were so small, unknown variations in conditions in different parts of the slab, such as the formation of cracks at certain parts, may influence the distribution of stresses more than some of the known elements of the design.

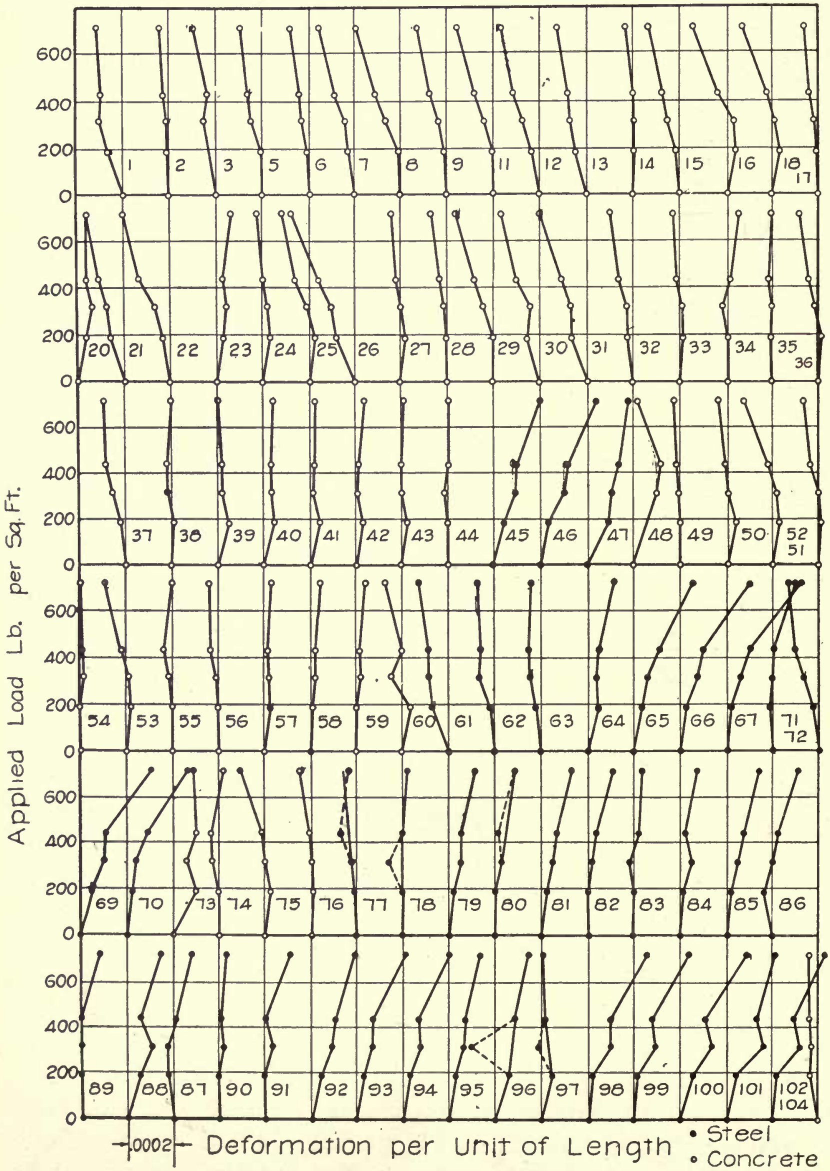


FIG. 50. LOAD-DEFORMATION DIAGRAMS FOR GAGE LINES 1 TO 102 OF TEST IN SCHULZE BAKING COMPANY BUILDING.

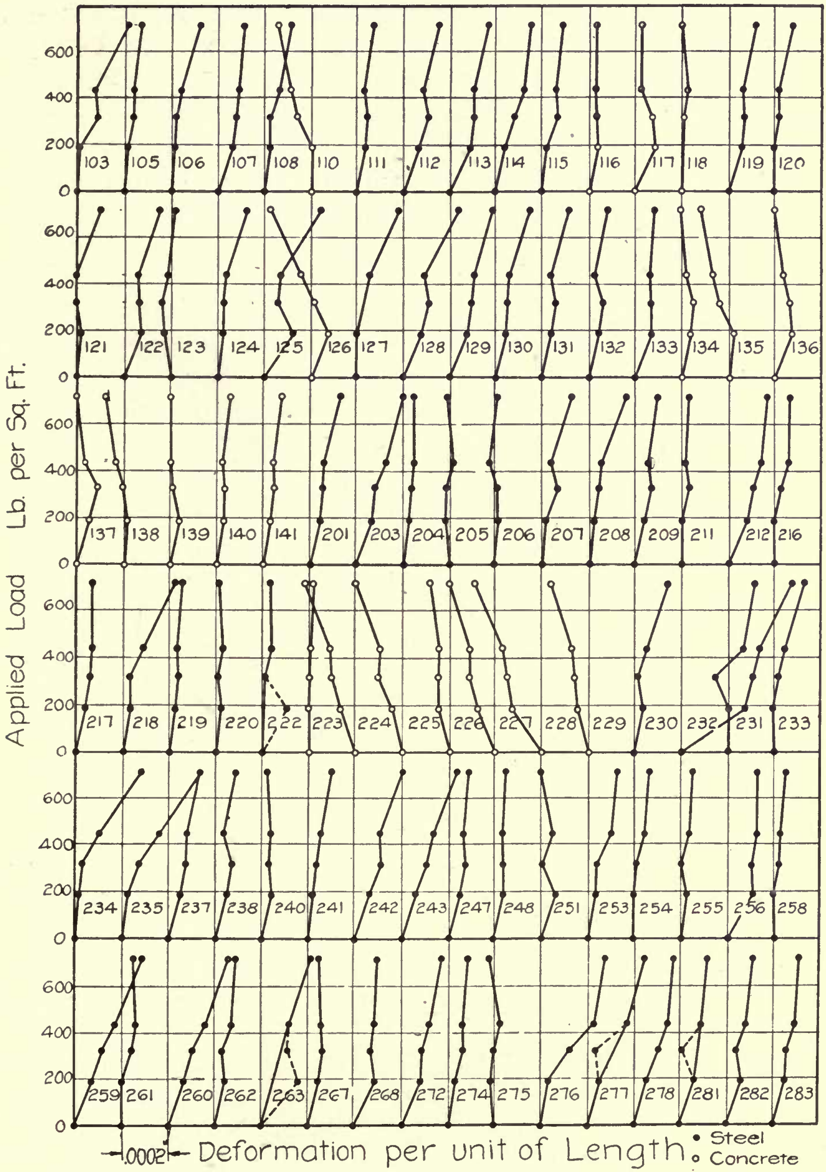


FIG. 51. LOAD-DEFORMATION DIAGRAMS FOR GAGE LINES 100 TO 283 OF TEST IN SCHULZE BAKING COMPANY BUILDING.

For instance, the formation of cracks at certain positions may be a more important factor in stress distribution than the difference in panel length in the two directions. In the use of this table such limitations must be kept in mind. Load-deformation diagrams, averages for groups of gage lines, are given in Fig. 49 and the diagrams for the individual gage lines are given in Fig. 50 and 51. Unit-deformations across or along various sections through the floor are shown in Fig. 52 to 55. The locations of the sections are shown in Fig. 44 and 45.

Table 7 indicates that the stress at the central column and the stress at the column on the edge of the loaded area were greater in the direction of the longer side of the panel than in the direction of

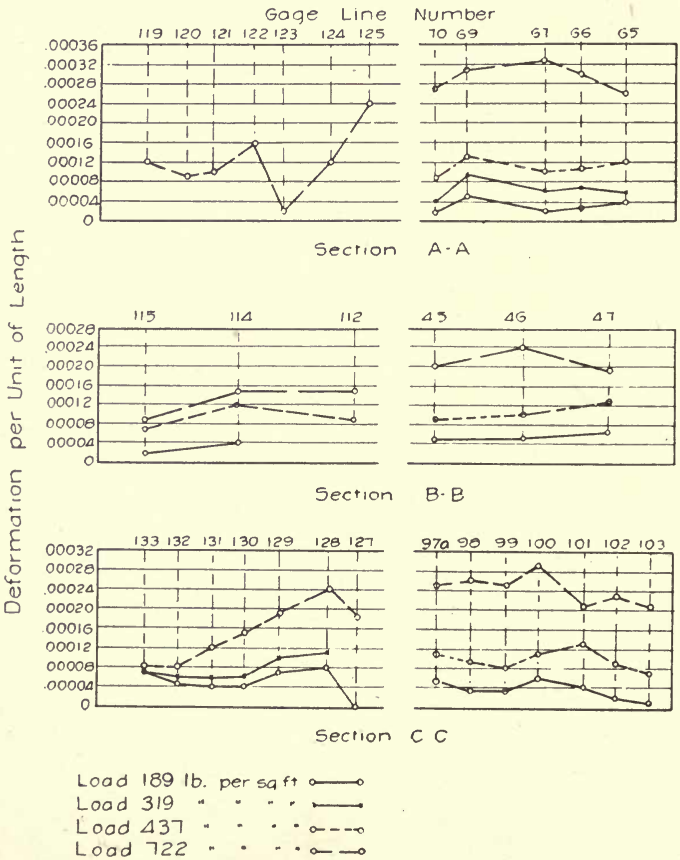


FIG. 52. DEFORMATION IN REINFORCEMENT IN RECTANGULAR BANDS MIDWAY BETWEEN COLUMNS IN TEST FLOOR OF SCHULZE BAKING COMPANY BUILDING.

the shorter side. For the center of the span a similar relation is shown.

A comparison of the deformations at the left hand portions of the sections shown in Fig. 52 with those at the right hand portions shows that the average stress in the rectangular band at the edge of the loaded area was considerably less than that in the corresponding band at the central portion of the loaded area. There is also an indication that the stress increased (somewhat irregularly) from the outer portion of the outer band at the edge of the loaded area toward the inner edge of this band.

In the diagonal bands of reinforcement the stress was about the mean of the average stresses in the two rectangular bands for positions at the central column and at the center of the panel (see Table 7). However, the stress in the diagonal band at the corner of the loaded area was greater than that in the rectangular bands at the columns on the edges of the loaded area. Only a few gage lines were read at the positions near the edge of the loaded area and the stresses found there may not represent the normal conditions for such positions.

The measurements on gage lines 230 and 231 (taken on the bars placed across the boundary line between two panels) indicate an average stress of about 3000 lb. per sq. in. at the maximum load (see Fig. 51). This would indicate that such rods may be effective in accomplishing the purpose for which they were designed, namely, the distribution or prevention of cracks which are likely to occur along the line from column to column. It indicates also that reinforcing bars in such positions may be of considerable value as reinforcement for negative moment. Of course, the reinforcing bars should be carried well beyond the usual position of the point of inflection to provide sufficient anchorage for cases of partial loading.

26. *Compressive Stresses and Unit-Deformations.*—Table 8 gives average values of the compressive unit-deformation in the concrete for representative positions. In tests of girderless slabs high unit-deformations are expected on the bottom of the depressed head close to the capital, and those on the bottom of the thin portion of the slab close to the edge of the depressed head have been found to be nearly as high. Examination of the data of this test shows that in the rectangular directions the deformations in these two places are not far different from each other.

The largest compressive unit-deformation, 0.00028, was found in the diagonal direction at the central column on gage line 26. Using a value of 3,000,000 lb. per sq. in. for the modulus of elasticity of

TABLE 8.

AVERAGE COMPRESSIVE UNIT-DEFORMATION AT MAXIMUM LOAD.

Gage Lines	Location of Gage Lines	Unit-deformation
8, 11, 16, 21, 25, 31	Long rectangular direction on bottom of depressed head	.00016
22, 32, 37	Short rectangular direction on bottom of depressed head	.00013
13, 61, 72	Long rectangular direction on slab near depressed head	.00012
21, 32, 36	Average of long and short rectangular directions near column capital	.00011
61, 138	Average of long and short rectangular directions near depressed head	.00013
9, 22, 31, 37	Across panel lines, bottom of depressed head near edge	.00015
*224 to 229 inclusive	Long rectangular direction midway between columns	.00015

*On upper surface of slab; all others on under surface.

concrete (frequently assumed in such cases), the corresponding compressive stress is 840 lb. per sq. in. A relatively large unit-deformation is found at the other end of the same diagonal in gage line 126. All other compressions in the diagonal direction were small.

In the long rectangular direction it is found that the maximum deformations would be cut by a curved section through gage lines 8, 11, 16, 21, 25, and 31 (see section J-J, Fig. 45). The average unit-deformation across this section was 0.00016 (see Table 8). In the short rectangular direction the average unit-deformation on gage lines 22, 32, and 37 was 0.00013.

It is seen that in general the deformations in the concrete were low. With the concrete intact in tension the neutral axis would be farther from the compression face than it would be otherwise. For the same resisting moment this would result in a smaller compressive

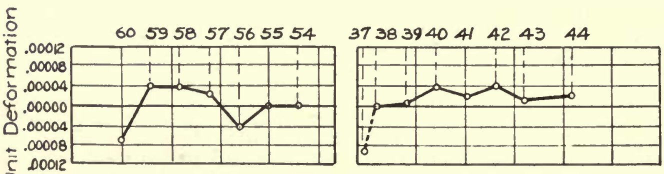


FIG. 53. LATERAL DISTRIBUTION OF COMPRESSIVE DEFORMATION ON BOTTOM OF TEST FLOOR NEAR DEPRESSED HEAD IN SCHULZE BAKING COMPANY BUILDING.

stress in the concrete than if the concrete had cracked generally on the tension side.

27. *Lateral Distribution of Compression.*—Little compression was detected in the two groups of gage lines 54 to 60 and 38 to 44 (see section F-F, Fig. 45 and 53). In fact, some of the measurements indicate a slight tension, but this is so small that the evidence should not be taken as conclusive. This indicates that the moment arm of the tensile stresses in the concrete in the thin portion of the slab may be determined as much by the thickness of the slab within the depressed head as by the thickness of the thinner portion of the slab. The tensile resistance of the concrete in the thinner portion of such a slab may thus give an increased resisting moment over that for a slab without the depressed head. This result suggests also that with a depressed head of the thickness here used the neutral axis is likely to be so close to the level of the bottom of the thin portion of the slab that the section of the slab outside of the depressed head may not be considered to contribute very much to the compressive stresses of the negative resisting moment. Tests to be used for comparing slabs having depressed heads with slabs which do not have them should employ loads large enough to eliminate effects of the kind here discussed.

Fig. 54, section G-G, shows the distribution of the compression

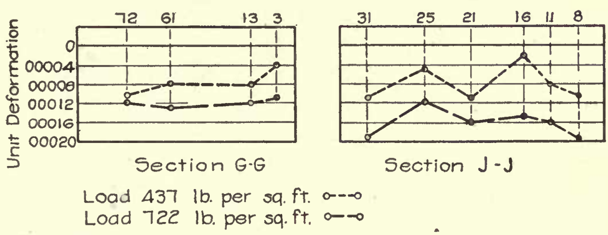


FIG. 54. COMPRESSIVE DEFORMATION ON BOTTOM OF TEST FLOOR NEAR DEPRESSED HEAD, AND NEAR COLUMN CAPITAL IN SCHULZE BAKING COMPANY BUILDING.

which is developed across a section close to the capital, and J-J shows the distribution of compression which is developed across a section in the thin portion of the slab close to the edge of the depressed head.

28. *Moment Coefficients.*—Assuming that all the load is carried by flexure, it is apparent that moment coefficients calculated on the basis of the stress in the steel only, will not represent the full resisting moment if the concrete assists in carrying the tensile stresses. In this test the tensile stresses were so low that it is certain that the concrete must have assisted greatly. However, while values of the

moment coefficient obtained by using the observed stress in the steel cannot be used for design, a comparison of the coefficients obtained for different parts of the panel may be of interest.

The resisting moments of the stresses in the reinforcement as given by calculations for the load of 722 lb. per sq. ft. are 0.0050 Wl for the positive and 0.0097 Wl for the negative moment, in which W is the total applied load on the panel and l is the average of the long and the short panel length center to center of columns. These are the components, in a direction parallel with a panel side, of the moment resisted by the stresses in all the reinforcement cut by a section across the entire panel at a position of maximum negative moment and at one of maximum positive moment. The values given are averages of the moment found for the long direction and the short direction of the panel.

Since no cracks could be found in the concrete it is certain that the tensile stresses in the concrete must have assisted very greatly in resisting the bending moment. The bending moment coefficients given, therefore, are not values which may be used for design purposes, and as the concrete may have given more assistance relatively at one of these positions than at the other, the values may not even give the correct ratio between the negative bending moment and the positive bending moment.

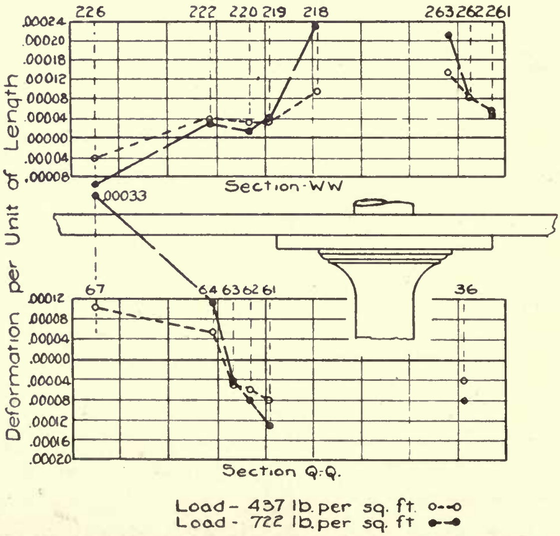


FIG. 55. LOCATION OF POINTS OF ZERO UNIT-DEFORMATION ON UPPER AND UNDER SURFACES OF TEST FLOOR OF SCHULZE BAKING COMPANY BUILDING.

29. *Point of Inflection.*—Points of zero unit-deformation on the upper and under surfaces of the slab in the direct lines between columns may be taken from the diagrams showing unit-deformation along sections W-W, Q-Q, L-L, and R-R, Fig. 55 and 56. These data

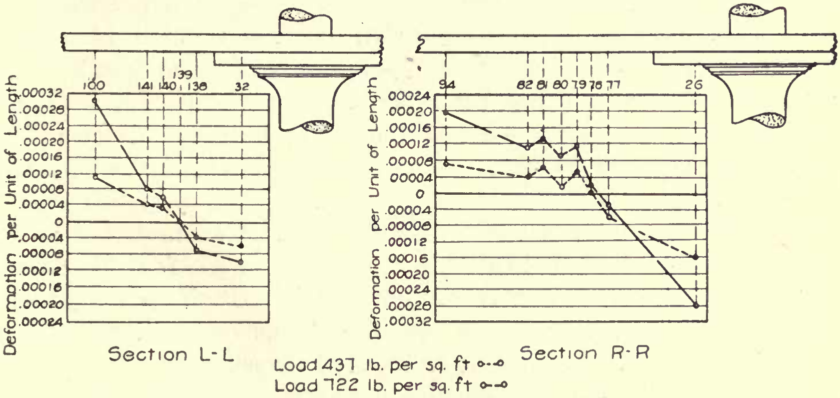


FIG. 56. LOCATION OF POINTS OF ZERO UNIT-DEFORMATION ON UNDER SURFACE OF TEST FLOOR OF SCHULTZE BAKING COMPANY BUILDING.

are inconclusive, but the indications are that as in other tests reported in this bulletin the point of zero unit-deformation for the under side of the slab was closer to the column than that for the upper side; or, in other words, the total tensile stress on any section seems to be greater than the total compressive stress. The presence of arch action would imply an excess of compression. This phenomenon has been observed in several tests and its occurrence need not be doubted. No explanation of this matter is attempted.

30. *Columns.*—The deformations in the columns were rather erratic and allow interpretations of only the most general sort. The differences in amount and character of deformations on opposite sides of the same column were sufficient to make it clear that there was considerable bending of the columns. The largest compressive deformation was found in column 2 which is on the edge of the loaded area, and the largest difference between deformations on opposite sides of any column is found for column 9 at the corner of the loaded area.

The significance of these observations seems to lie in their indication that the most important element of the deformation developed in the columns by load tests is that caused by the moment of unbalanced loads and that there is a larger amount of flexure in the corner columns than in the columns at the side of the loaded area. The latter is indicated also by the observation that the stress was greater in the diagonal band of slab reinforcement at the corner of the loaded area than in the rectangular bands at the edge of the loaded area.

31. *Examination for Cracks.*—A careful examination of the surfaces of the slabs and columns disclosed no cracks. This was unexpected and it is the only case known personally to the writers in which a floor loaded to twice the design live load plus the dead load did not develop cracks which were large enough to be found by a reasonably careful examination. The absence of visible cracks on the shaft of the columns may be due to the influence of the dead weight of the floors above in overcoming a portion of the flexural tension. However, at certain portions of the slab it seems probable that there were very minute cracks because in several instances steel stresses in the neighborhood of 10,000 lb. per sq. in. were found. Even though there may have been very minute cracks a considerable portion of the concrete must have been intact, and this undoubtedly had a great deal to do with keeping down the steel stresses.

32. *Deflections.*—Load-deflection diagrams are shown in Fig. 57.

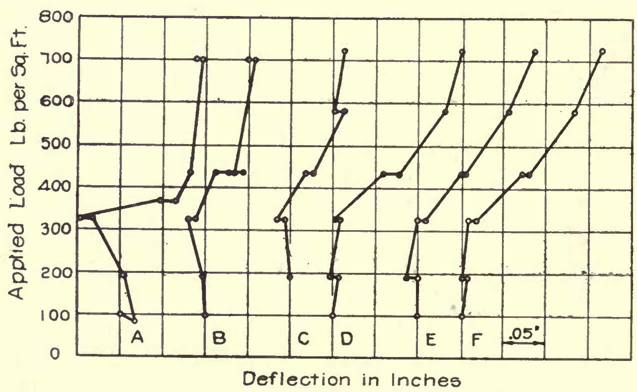


FIG. 57. LOAD-DEFLECTION DIAGRAMS FOR FLOOR TEST IN SCHULZE BAKING COMPANY BUILDING.

Distance to the right of the zero line represents downward deflection. The first set of readings was not obtained until a load of about 100 lb. per sq. ft. was on the floor and at deflection point C the apparatus was disturbed after the first set of readings had been taken; hence the deflection cannot be shown below 190 lb. per sq. ft. for C nor below 100 lb. per sq. ft. for all other points. The indications are that the failure to get readings for zero load makes little difference in the appearance of the curves. The small deflections and the tendencies toward upward deflection for the loads below 319 lb. per sq. ft. are ascribed to the rather extreme temperature changes. The air temperature rose from 20° F. at the time the load of 319 lb. per sq. ft. was placed to 50° F. at the time the load of 437 lb. per sq. ft. was placed, and then fell again to 25° F.

33. *Summary of Results.*—The results of the test here reported are conditioned upon a correct interpretation of the effect of the rather extreme temperature variation during the time of the test and of the assistance given by the strength of the concrete in tension. A continuation of the test to a point at which the concrete in tension had cracked generally, probably would modify many of the conclusions.

The main results pointed out in the foregoing paragraphs or shown in the diagrams are:

1 Very few steel stresses higher than 6000 lb. per sq. in. were found. On only two gage lines did the observed deformations indicate steel stresses as high as 10,000 lb. per sq. in. The highest of these was 14,400 lb. per sq. in., but the form of the curve indicates that the initial reading may have been in error and that 10,000 lb. per sq. in. is a more probable value.

2 The averages of the stresses in the bands of reinforcement passing under the central portion of the loaded area were higher than averages in the bands under the edges of the loaded area. In the latter also the stresses in the inner bars of the band (the bars on the side toward the center of the loaded area) were larger than the stresses in the outer bars (lying outside the loaded area).

3 The stresses in the reinforcement of the diagonal bands fell between the averages for the two rectangular bands for positions around the central column and midway between columns. The stresses were larger in a diagonal band at the corner of the loaded area than in a rectangular band where it crosses the edge of the loaded area.

4 The stresses in the short bars placed across the panel boundary lines were low but large enough to indicate that the bars may be effective in distributing or preventing cracks along the edge of the panel.

5 The compressive unit-deformations were low.

6 The compressive deformations across a section of the slab for gage lines as near as possible to the edge of the depressed head were nearly as high as those across the section of the depressed head near the edge of the capital.

7 The largest compressive unit-deformation was found in the diagonal direction at the central column.

8 The portions of the slab beyond the edge of the depressed head did not develop compressive stresses on the under side in a direction parallel to that edge.

9 Moment coefficients calculated on the basis of the steel stresses developed are exceedingly low. That for a position of maximum negative moment is about twice as large as that for a position of maximum positive moment.

Especial emphasis should be placed on the fact that these coefficients cannot be taken as indicating the total resisting moment developed.

10 The indications are that the bending of the columns was an important feature of the action of the structure. The largest bending apparently occurred in a column at the corner of the loaded area.

V. The Worcester Slab Test.

34. *The Test Structure.*—The structure on which the test was made was built especially for the test and was located near Worcester, Mass. The slab was designed with the object of obtaining information on the effect of (1) different methods of arranging and distributing the reinforcement, and (2) variation in size of column capital. Fig. 58 gives the general design of the structure.

In order to avoid as far as possible lack of uniformity in conditions of building and testing the different parts and in order to reduce the proportion of the number of wall panels to interior panels, the four types of design used were placed in the four quadrants of a single slab four panel lengths (56 ft.) square. This gave a group of four panels to each of the four designs and a column in the center of each group. The details of the slab are shown in Fig. 58. The arrangements of slab reinforcement at the column capitals for the various groups were as follows:

Group I All tension reinforcement was placed in the diagonal bands. The rectangular bands lay in the bottom of the slab and afforded compression reinforcement at the column capitals.

Group II Both rectangular and diagonal reinforcement were in the top of the slab. There was no reinforcement in the bottom of the slab at the columns.

Group III All tension reinforcement was placed in the diagonal bands. The bars of rectangular bands did not pass over the column capitals. There was no reinforcement in the bottom of the slab at these positions.

Group IV Reinforcement was the same as in Group II but the column capital was smaller.

The amount and distribution of the tension reinforcement at the column capitals in each of the four groups were such that if planes were passed cutting each band at right angles outside the column capital the total area of steel in the top of the slab so cut in an angular distance of 180° around the column was 4.86 sq. in., the same for all groups. It should be noted, however, that the effect of the reinforcement in producing resisting moment across a panel edge will not be exactly the same for all groups. In the group having only diagonal bands a calculation of the rectangular component of the resisting moment (the component in a direction at right angles to the edge of the panel) will be about one-sixth greater than the rectangular component of the resisting moment of the reinforcement in the groups having both diagonal bands and cross bands. This area includes the section of tension reinforcement which in Groups II and IV consists of two rectangular bands and two diagonal bands; in Groups I and III this area is the section of two diagonal bands.

At points midway between columns, both in the rectangular and in the diagonal directions, the amount and distribution of the reinforcement were the same for all groups. In Group III the bars in the rectangular bands ended near the probable points of inflection and were not bent up in any way. The total area of cross-section of two rectangular bands and two diagonal bands was 4.2 sq. in.

Midway between columns two bars 3/8 in. in diameter, 6 ft. long, were placed in the top of the slab across the panel boundary, that is, normal to the direction of the rectangular bands of reinforcement.

The panel length was 14 ft. center to center of columns in all panels. The diameter of the top of column capital was 4 ft. 6 in. for Groups I, II, and III, and 2 ft. 9 in. for Group IV. The ratios of these capital diameters to the panel length were 0.321 and 0.196 respectively. The average of all the measured thicknesses of the slab was 4.93 in. The average measured depth to the center of gravity of

TABLE 9.

CALCULATED SOIL PRESSURE AND MEASURED SETTLEMENT FOR UNIFORM LOAD OF 215 LB. PER SQ. FT.

Column Numbers	Soil Pressure lb. per sq. ft.	Average Settlement in.
E1 A1 E5 A5	1600	.11
E2 D1 B1 A2 E4 D5 B5 A4	2300	.235
C1 A3 C5 E3	2000	.27
D2 B2 D4 B4	3000	.39
C2 B3 C4 D3	2700	.27+
C3	2400	.37

the bands of reinforcement was 1.04 in. midway between columns and 1.63 in. near the columns for the four groups.

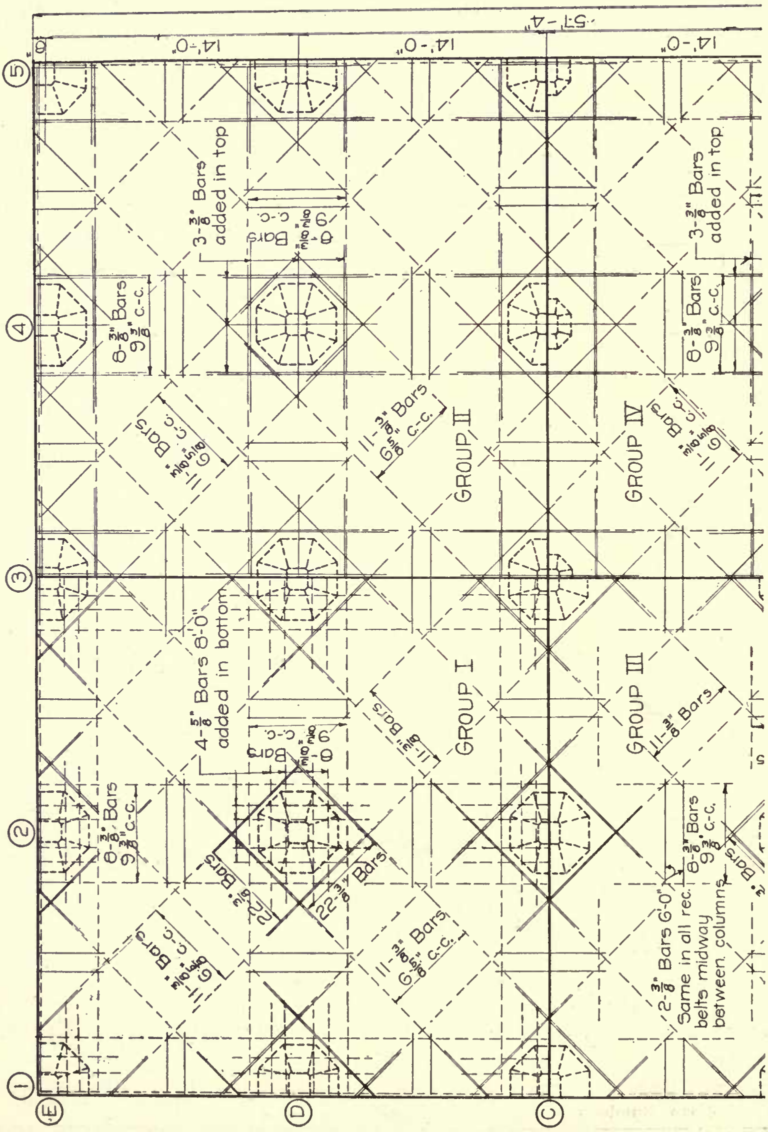
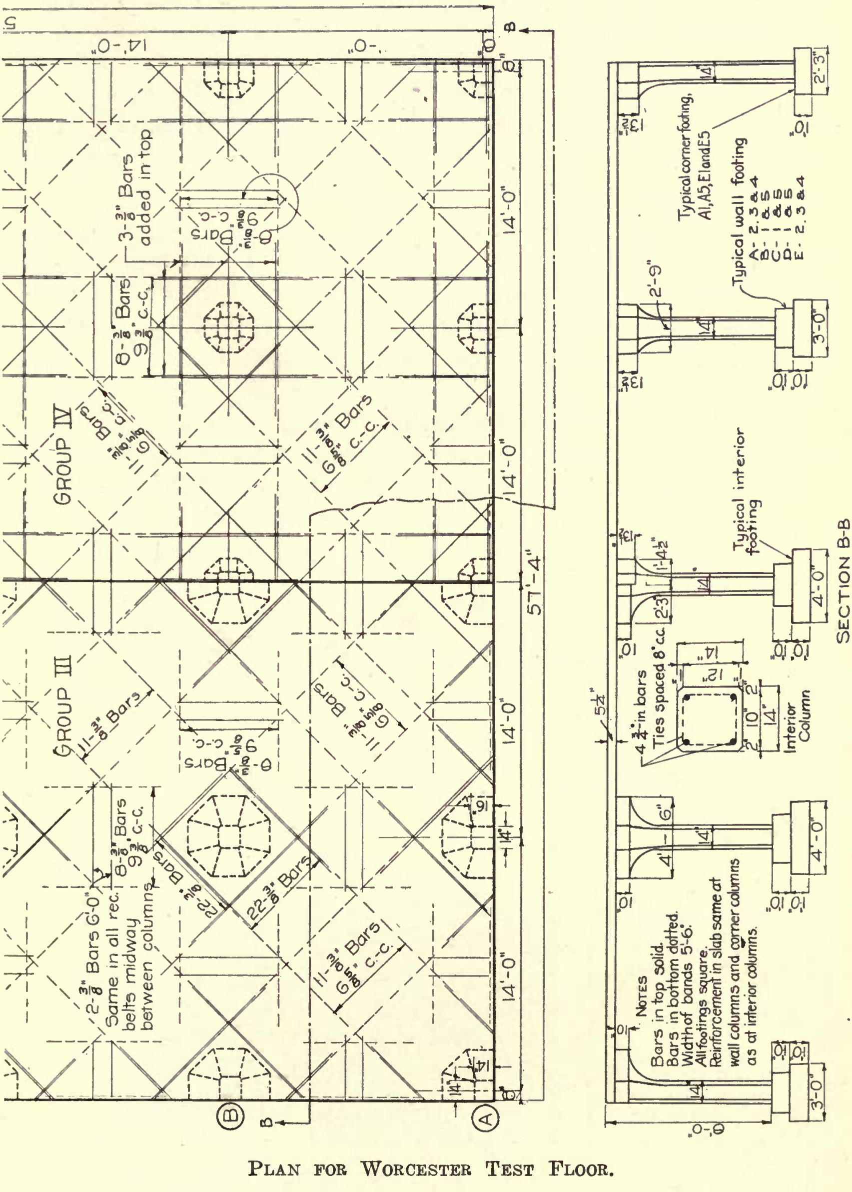


FIG. 58. DIMENSIONS AND REINFORCING

The footings for the interior columns were 5 ft. square, for the corner columns 3 ft. square, and for the columns at the edges 4 ft.



PLAN FOR WORCESTER TEST FLOOR.

square. It seems likely that the unit pressure on the soil was greater for the interior footings than for the footings at the corners and edges of the slab. Table 9 gives soil pressure calculated on the assumption that the total footing pressures were equal to the sums of the reactions of two systems of four-span, freely supported, continuous beams crossing each other at right angles.

35. *The Test.*—Preparations for testing were started on July 17, 1913, the loading began on July 28, 1913, and the last readings were taken on August 2, 1913. At the time of the test the concrete was about 3½ months old.

Gravel from the bank close at hand was used as loading material. The location of the slab on the side of the gravel hill allowed convenient access to loading material. A runway was built from the test slab to a point on the gravel bank somewhat above the elevation of the slab so that even at the higher loads little elevating of the loading material was necessary.

As explained in the following article, it was necessary to take strain gage readings early in the morning. This made it difficult to obtain readings with the load uniformly distributed over the slab. It was planned to make each increment of load such that it could be completed in a day, but these plans were interfered with by

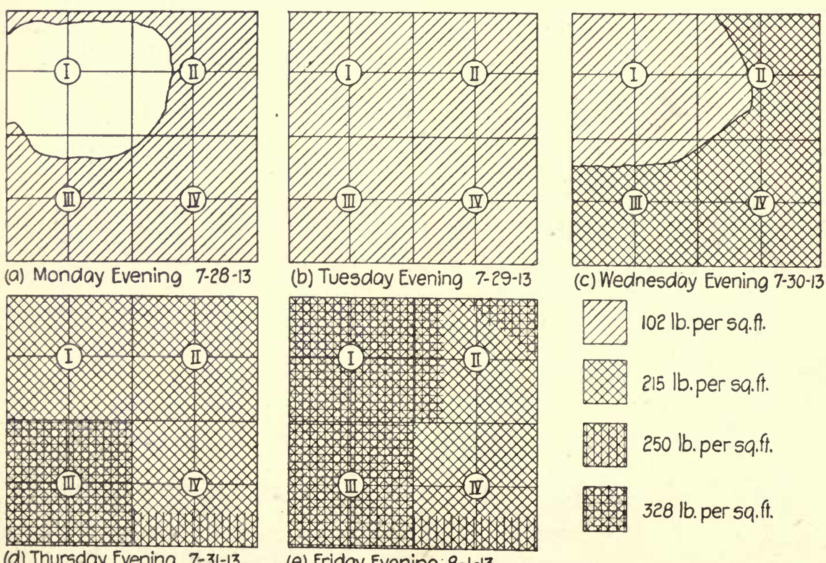


FIG. 59. PLAN SHOWING DISTRIBUTION OF LOAD OVER WORCESTER TEST FLOOR AT SUCCESSIVE STAGES OF THE TEST.

rainy weather, by labor troubles, and by the failure of a portion of the slab at a load between 215 and 250 lb. per sq. ft., a smaller load than it was expected to carry. Fig. 59 indicates the intensity of the load upon the various portions of the slab at various times. After taking the readings at the maximum load the test was discontinued, the slab being left with the load upon it.

Measurements were taken of deformation in the steel and in the concrete, of deflections at the centers of the four interior panels, and of settlement of footings.

Measurements of deformation were taken on 313 gage lines (see Fig. 60, 61, and 72). Of these 281 were on the slab and 32 were on the columns. Of the gage lines on the slab 113 were on the concrete and 168 on the steel. The strain gage observations were made by Professor H. F. Moore of the University of Illinois and Mr. Slater. Great care was exercised in obtaining initial or zero readings. On a predetermined set of gage lines each observer took two independent sets of strain gage readings. Then instruments and assignments of gage lines were exchanged and a third set of zero-load observations was taken. In this way three independent observations were taken on each gage line. Time was taken then to compare the corrected observations before proceeding with the loading. It was found that there was serious lack of agreement not only between the check readings taken by the two observers but also between check readings taken on the same gage line by either observer, the amount of the discrepancy depending somewhat on the time of day the observations were taken.

Evidence was found indicating that temperature variation throughout the day was responsible for the discrepancies between check readings and that the discrepancies could be reduced greatly by taking readings early in the morning. Accordingly, the first three series of zero-load readings were disregarded and one new series was taken by each observer on all the 313 gage lines, beginning at 4:20 a. m. August 28 and finishing a little before 9 a. m. of the same day. The readings thus obtained gave satisfactory checks on accuracy of observation. All subsequent strain gage readings except those used for special purposes were taken between about 4 a. m. and 7 a. m. This experience is given in some detail because it suggests some of the difficulties attending the carrying out of experimental work of this character.

In order to obtain information which might be expected to have the widest range of applicability without undue expenditure of time,

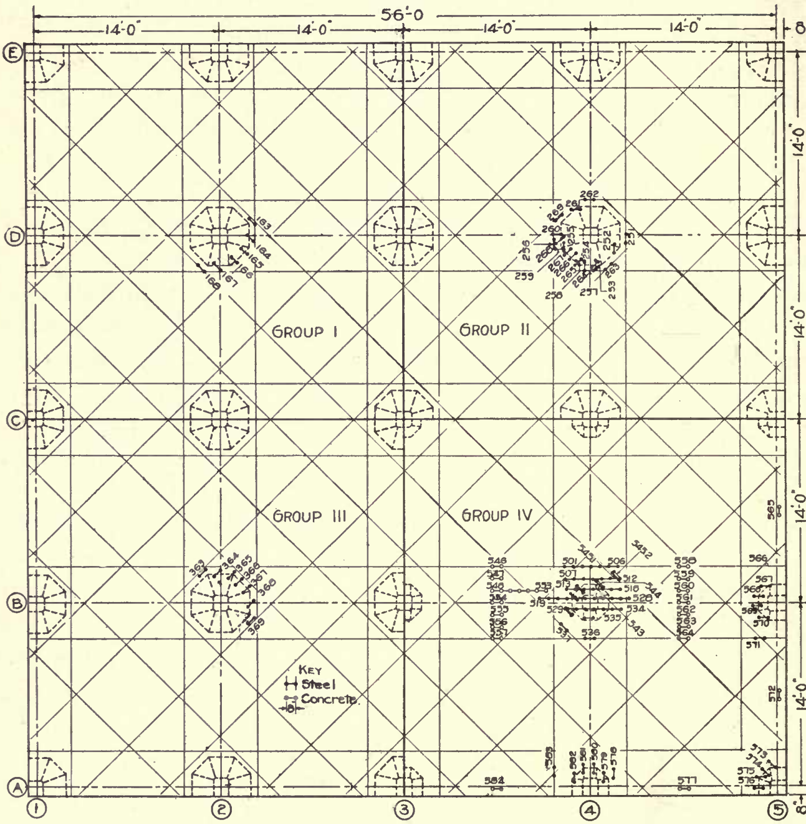


FIG. 60. LOCATION OF GAGE LINES ON UPPER SURFACE OF WORCESTER TEST FLOOR.

it was decided to use a large number of gage lines in one group and a smaller number in each of the other groups. It was expected that the ratios of stresses in interior panels to those at similar positions in wall panels would be about the same for all groups so that a detailed study of one panel would allow the missing parts of the data of the other panels to be filled in. Group IV was chosen as being the most suitable for this purpose because with its smaller capital it would be expected to show higher stresses and because the size of its capital seemed to be nearer general practice than was that of the other capitals. The results of the test seem to indicate that the assumption as to comparative values of stress in similarly located positions was not justified. This probably was due largely to the complications caused by the settlement of columns.

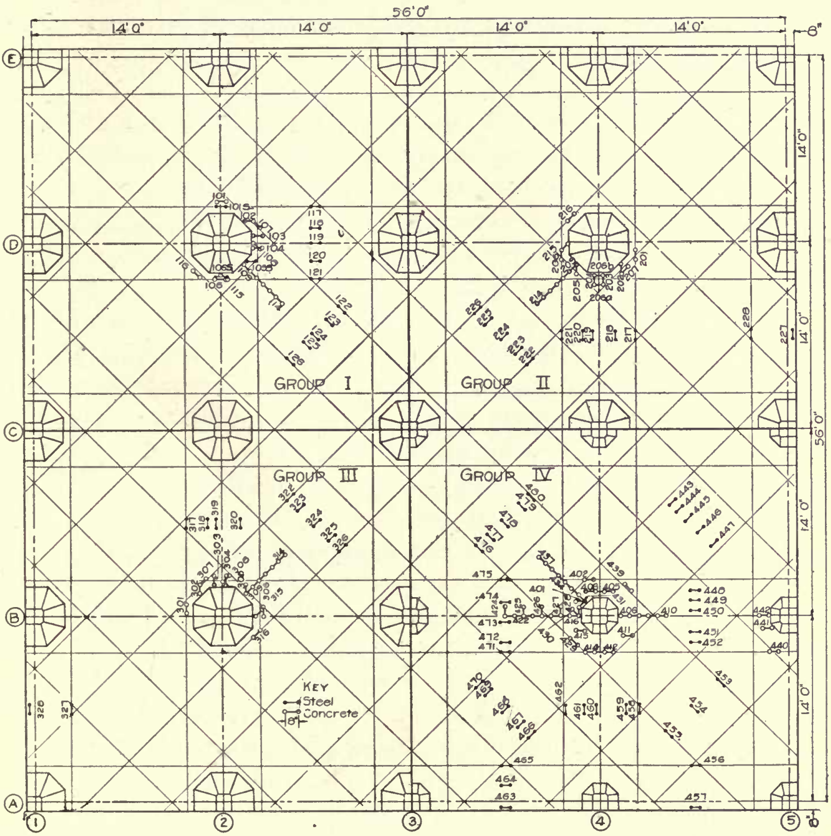


FIG. 61. LOCATION OF GAGE LINES ON UNDER SURFACE OF WORCESTER TEST FLOOR.

Deflections were measured at the center of the inner panel of each of the four groups, but these are so affected by the large settlement of column footings that slight importance is attached to them.

Settlement of footings was measured. A large amount of settlement, of course, was not anticipated and it was thought at first that measurement of settlement of the footings at the corners and at the centers of each of the four groups would be sufficient. These measurements were taken by means of a deflectometer (see Fig. 17a, Bulletin 64 of the University of Illinois Engineering Experiment Station). When a load of 100 lb. per sq. ft. had been applied to the slab it was found that the settlement was so large as not to justify so great refinement of measurement, and the difficulty of keeping the apparatus in adjustment in the presence of so many laborers led to the abandonment of this method and to the use of an engineer's level as a means of measuring further settlement. Average amounts of settlement are shown in Table 9.

36. *Phenomena of the Test.*—The application of load was begun on Monday morning, July 28, 1913. The first increment of load, approximately 100 lb. per sq. ft. over the entire slab, was completed on July 29. Strain gage readings for the load of 100 lb. per sq. ft. were taken from 4:00 to 8:00 a. m. on July 30. At this load a crack was observed across gage line 581 (at the edge of the capital of the wall column), the stress at this place being 9000 lb. per sq. in. No other cracks were recorded for this load, but stresses in the reinforcement were nearly 10,000 lb. per sq. in. in several gage lines at the center column of Group IV on the diagonal band; also at the corner and wall columns of the same group. Therefore, it seems nearly certain that other cracks were present at such places.

At the load of 100 lb. per sq. ft. it was found that there was considerable settlement of the footings.

On July 30 additional load was applied to Groups II, III, and IV, bringing the load on this area up to 215 lb. per sq. ft. while Group I still had only 100 lb. per sq. ft. Strain gage readings were taken on July 31 from 4:00 to 7:00 a. m. with this load in position. This load is referred to later as the load of 215 lb. per sq. ft. and its distribution is shown in Fig. 59.

At this stage of loading cracks appeared on the outer surface of column D5 across gage lines 8 and 9, Fig. 73, which were on the column reinforcement at the lower part of the bell-shaped portion of the column. The unit-deformation in gage line 9 across the lower crack corresponded to a stress of 46,000 lb. per sq. in. In gage line 10, which was at the bottom of the bell-shaped portion of the column and below the cracks, so great an elongation had taken place that the reading was beyond the range of the instrument. A rough measurement indicated a unit-deformation at this place of 0.006. These high elongations indicate that at this stage of the test, however much slip of column bars there may have been, the cracks were in part due to the very high stresses in the reinforcement.

At the same stage of the test, at the central columns of the groups and midway between the columns the stresses in the slab reinforcement quite generally had reached 15,000 to 20,000 lb. per sq. in. The stresses in the slab reinforcement at the wall columns and at the outer corner columns of Group IV were much higher. The deformations at the latter places corresponded to from 40,000 to 50,000 lb. per sq. in. in several instances, a value beyond the yield-point of the steel. The highest deformations in the slab reinforcement were in the diagonal band at column A5, and scaling of one of the bars at this place was noted, indicating that the bar had been stressed to the yield point.

At a load of a little more than 100 lb. per sq. ft. a crack had appeared on the bottom of the slab of Group IV, describing approximately a quadrant of a circle having a radius of about 6 ft. with column A5 in the center of the circle. This crack was fairly large when first noticed and increased in size quite rapidly as the loading progressed. After completing the strain gage readings for the load of 215 lb. per sq. ft. on Group II, III, and IV more gravel was placed upon Group I, bringing the load up to 215 lb. per sq. ft. over the entire four groups. No readings were taken at this stage but another increment was begun by applying load to Group IV in the position indicated in Fig 59d. About one-fourth of Group IV had been loaded to an intensity of 250 lb. per sq. ft. when failure of the slab in this group occurred, allowing the slab at the center of the corner panel to deflect about 8 in. and to settle down upon the 4 by 4-in. post which had been used as a datum for the measurement of deflections. Except for the presence of this post it seems likely that the slab in Group IV would have fallen at this time. Large tension cracks appeared at the middle of the exterior corner panel and over the edges of several of the column capitals of this group, and there were appearances of diagonal tension failure around column B4, but it seems likely that this was a result, or at most a secondary cause, of failure. That this is true is indicated by high deformations at various places under the load of 215 lb. per sq. ft. Some bending of column A5 was visible, although the column had shown no structural defects. Column B5 showed structural defects previous to applying the load and high stresses were developed in the reinforcement of the slab near this column, but the column did not fail. The defects appeared to be due mostly to the first pouring of the concrete having been carried too high on the column. A portion of the bell was filled with the first pouring, and the inability of the concrete in the expanded portion of the column to follow down into the shaft of the column, as shrinkage developed with setting, appears to have left voids in the concrete at the place where the bell begins. Some other columns showed the same features, but to a smaller extent than in the case of column B5.

The area and intensity of the load on Group IV at the time of its failure is shown in Fig. 59d. No more load was applied to this part of the slab and with continued application of load in the other groups the signs of distress in column D5, noted in a previous paragraph, increased. Slipping of the concrete at the top of the column along the column reinforcing bar took place and the crack across gage line 8 opened greatly. At the same time bending in column E5 near the junction of the shaft with the bell became visible and a construction joint opened (see Fig. 73). When about three eighths of Group II was covered with 328 lb. per sq. ft., the remainder of the slab having only 215 lb. per sq. ft., the condition of this portion of the structure became critical and loading was discontinued.

The distribution of the maximum load over the entire slab is shown in Fig 59c. Yielding of column D5 seems to have brought about the critical condition existing in Group II at the maximum load. This column did not fail completely but it had yielded to such an extent that a heavy stress was thrown into the reinforcement extending from this column to column D4.

The settlement of the footings, which was observed to have begun before the completion of the 100-lb. per sq. ft. load, continued, and at the higher loads the difference in elevation of certain parts of the slab could be observed by sighting along the under sides of the column capitals. At the maximum load the settlement of column D4 was 2 1/16 in. Other columns had settled appreciably but no other so much as column D4, and there was no uniformity in the amount of settlement for the various columns.

37. *Usefulness of Results of Test.*—The large amount of settlement of the footings and its unevenness throw serious complications into the interpretation of the results of the test. It seems probable that the distribution of stresses may have been dependent as much upon the relative amounts of settlement of the various footings as upon the variation in distribution of the reinforcement. In the following paragraphs comparisons are given of the stresses and deformations found at various positions on the slab and the discussion of the effect of certain features of the design on the stresses. Such comparisons must be considered to be qualitative and not to show quantitative variations, and further tests may show errors in conclusions based upon such comparisons.

Notwithstanding the limitations to the usefulness of the data it is believed that a presentation of the observed deformations is of value. These are shown in Fig. 62 and 63.

38. *Effect of Variation in the Distribution of Reinforcement.*—An examination of Table 10 shows the same average steel stress in the slab at the central column of Group I as that at the central column of

TABLE 10.

AVERAGE STRESSES IN TENSION REINFORCEMENT; COMPARISON OF GROUPS.

These stresses are based upon an assumed modulus of elasticity of 30 000 000 lb. per sq. in.

Group	Location and Direction of Gage Lines		Gage Lines	Stress in Reinforcement lb. per sq. in. Load 102 lb. per sq. ft.	Load 215 lb. per sq. ft.
I	Midway between Columns	Rect.	117 to 121	5800	24700
		Diag.	122 to 126	1000	5200
	At Central Column	Rect.			
		Diag.	163 to 168	4300	8500
II	Midway between Columns	Rect.	217 to 221	5100	25700
		Diag.	222 to 226	2800	7300
	At Central Column	Rect.	251 to 256 and 257 to 262	3400 2400	7500 4500
		Diag.	263 to 268	4500	8500
III	Midway between Columns	Rect.	317 to 320	6700	14600
		Diag.	322 to 326	4000	8500
	At Central Column	Rect.			
		Diag.	363 to 368	5600	15400
IV	Midway between Columns	Rect.	*471 to 475	4000	18000
		Diag.	476 to 480	3900	10500
	At Central Column	Rect.	503, 509, 514, 522, 529, 535, 536	4200	23700
		Diag.	537, 538, 539, 540, 542, 542a, 543, 544, 545.1, 545.2	7200	26500
Ratio of stresses in group IV to average stresses in groups I, II, and III	Midway between Columns	Rect.		0.70	0.83
		Diag.		1.53	1.50
	At Central Column	Rect.		1.45	3.86
		Diag.		1.51	2.41

*Area immediately over these gage lines not fully loaded. See observation boxes, Fig. 74.

Group II at 215 per sq. ft., and nearly the same in the two locations at 102 lb. per sq. ft. This does not indicate that the difference in distribution of reinforcement had much effect on the steel stress. Likewise a comparison of the slab stresses and deformations on the upper and under sides of the slab at the columns of Groups I, II, and III (see Tables 10 and 11) shows little difference which it is thought can be attributed to the fact that Group I had compression reinforcement while Groups II and III had none.

39. *Effect of Variation in Size of Capital.*—A comparison of the stresses in the steel and the deformations in the concrete at the central capital of Group II with those at corresponding positions in Group IV may be expected to show the effect of the difference in the size of capi-

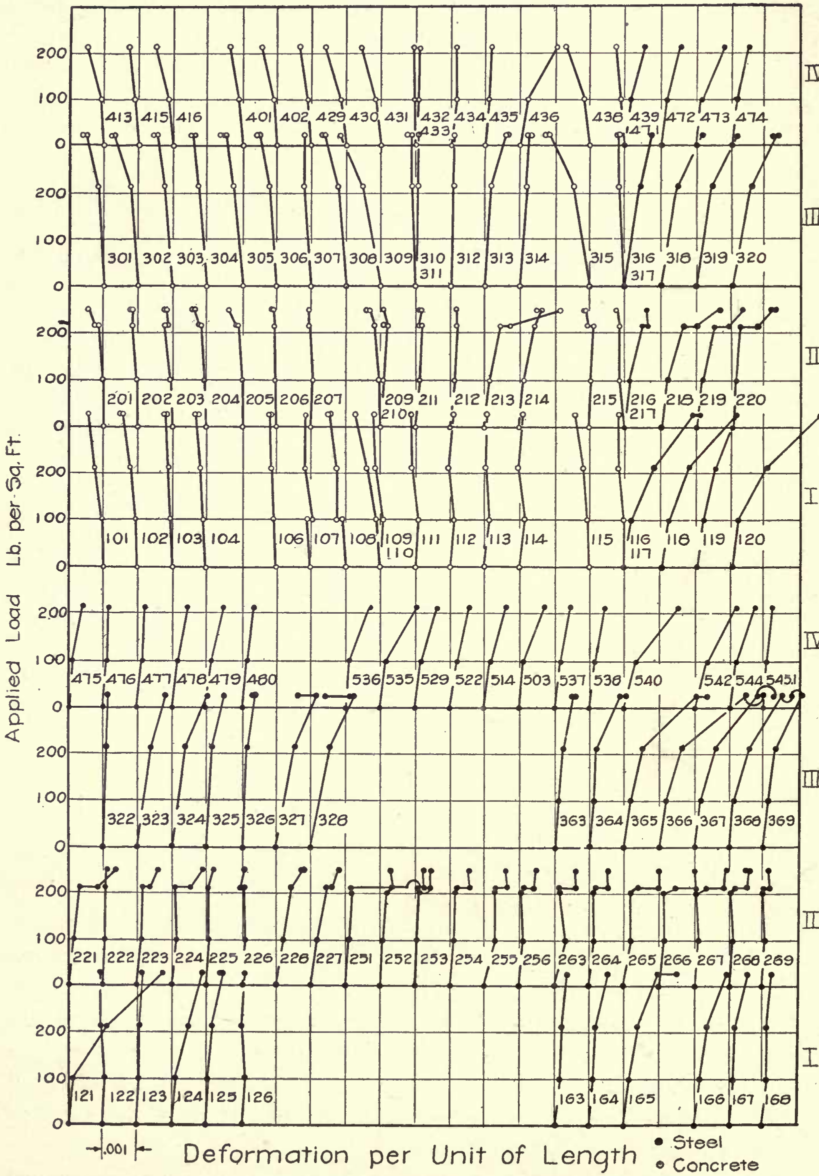


FIG. 62. LOAD-DEFORMATION DIAGRAMS ARRANGED FOR COMPARISON OF GROUPS IN WORCESTER TEST FLOOR

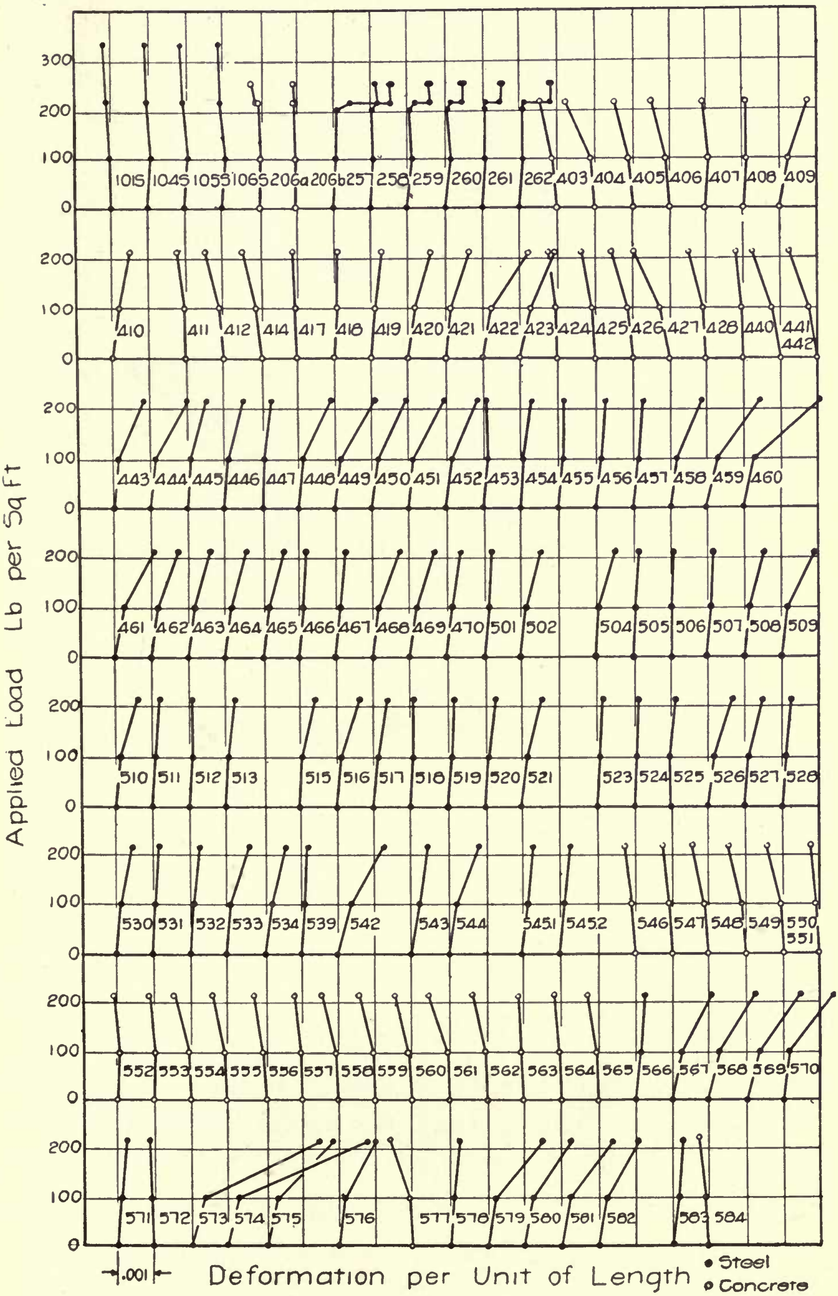


Fig. 63. Additional Load-deformation Diagrams for Gage Lines of Worcester Test Floor.

TABLE 11.

AVERAGE COMPRESSIVE UNIT-DEFORMATIONS IN CONCRETE; COMPARISON OF GROUPS.

Group	Location and Direction of Gage Lines		Gage Lines	Unit-deformation	
				102 lb. per sq. ft.	215 lb. per sq. ft.
I	At Central Column	Rect.	101 to 106	.000048	.000187
		Diag.	107, 108, 109, 115, 116	.000045	.000258
II	At Central Column	Rect.	201 to 206	.000038	.000197
		Diag.	207, 208, 209,* 216	.000063	.000099
III	At Central Column	Rect.	301 to 306	.000133	.000287
		Diag.	307, 308, 309, 315, 316	.000210	.000429
IV	Midway between Columns	Rect.	546 to 557	.000067	.000333
	At Central Column	Rect.	404, 401, 416, 415, 413	.000135	.000461
		Diag.	429, 430, 431, 438, 439	.000188	.000568

*215 indicated tension and was omitted.

tals since the design of these groups was the same in other respects. Such a comparison shows that the stresses and deformations were considerably larger in the case of the smaller capital (see Tables 10 and 11 and Fig. 62). A comparison of Groups II and III with Group IV shows nearly the same results.

Attention may well be called to the width and thickness of the large capitals which approach the dimensions sometimes given to the depressed head used over column capitals. It seems evident that the conditions are different from those of the ordinary column capital.

It is seen that the stresses in the steel in Group IV at the 102-lb. load were about 50 per cent greater than the corresponding average stresses in the other groups except at the center of the rectangular band in which case it was 30 per cent less.

The load of 102 lb. per sq. ft. was used for this comparison because at the higher load (215 lb. per sq. ft.) Group IV was on the point of failure, and the excessive stresses at this load are believed to be due partly to a yielding of certain parts of the slab, which apparently had already taken place.

40. *Comparison of Interior Panels with Exterior Panels.*—An examination of Table 12 indicates a smaller stress in the diagonal at the center of the corner panel of Group IV than at the center of the interior panel. In this connection it will be recalled that the corner

TABLE 12.

AVERAGE STRESSES IN TENSION REINFORCEMENT MIDWAY BETWEEN COLUMNS IN GROUP IV; COMPARISON OF INTERIOR PANELS WITH EXTERIOR PANELS.

By term corner panel is meant a panel having two exterior edges.
By term wall panel is meant a panel having one exterior edge.
By term interior panel is meant a panel having no exterior edges.

Band	Location and Direction of Gage Lines	Gage Lines	Stress in Reinforcement lb. per sq. in.	
			Load 102 lb. per sq. ft.	Load 215 lb. per sq. ft.
Diagonal	Corner panel	453, 454, 455	3100	5600
Diagonal	Wall panel	443 to 447 466 to 470	3700 4600	18600 14900
Diagonal	Interior panel	476 to 480	3900	10400
Rectangular	Between corner panel and wall panel	*448 to 452 458 to 462	3800 8100	29200 40800
Rectangular	Between wall panel and interior panel	*471 to 475	4100	18000
Rectangular	Exterior edge of wall panel	463, 464, 465	5900	17900
Rectangular	Exterior edge of corner panel	456, 457	4500	8500

*Area immediately over these gage lines not fully loaded. See observation boxes, Fig. 74.

panel was the first to show distress by the appearance of a large crack across the diagonal band of reinforcement. The average of the steel stresses in the diagonal bands at the center of wall panels of Group IV was larger than the corresponding stresses in the interior panel for loads of 102 and 215 lb. per sq. ft. In the rectangular bands lying between wall panels and corner panels at points midway between columns the steel stresses were considerably greater than the stresses in the bands between the wall panels and interior panels. These comparisons are for cases where bending of columns at the edge or corner of the slab may be expected to influence the amount of the moment carried by the slab at the center of the span. The conclusion seems justified that the bending of the wall columns was sufficient to allow a material increase in the moment near the center of the rectangular span. The position and size of the cracks indicate that the proportional increase in the moment near the center of the diagonal in the corner panel was still larger though the location of the gage lines was such as not to show that this was true.

41. *Stress at Exterior Edge of Slab.*—Tables 13 and 14 were prepared to show a comparison of stresses and unit-deformation at the exterior edge of a panel with those at similar positions on an interior

TABLE 13.

AVERAGE STRESSES IN TENSION REINFORCEMENT; COMPARISON OF EXTERIOR RECTANGULAR BANDS WITH INTERIOR RECTANGULAR BANDS.

Group	Location of Gage Lines	Gage Lines	Stress in Reinforcement lb. per sq in.	
			Load 102 lb. per sq. ft.	Load 215 lb. per sq. ft.
II	Outer edge wall panel	227, 228	6800	17500
II	Between wall panel and interior panel	217 to 221	5100	26200
III	Outer edge corner panel	327, 328	*1800	18000
III	Between wall panel and interior panel	317 to 320	*3600	14500
IV	Exterior edge corner panel	456, 457	4500	8500
IV	Exterior edge wall panel	463, 464, 465	5900	17900
IV	Between corner panel and exterior panel	448 to 452 458 to 462	3800 8100	29200 40800
IV	Between wall panel and interior panel	471 to 475	4100	18000

*No values tabulated; these obtained from load-deformation curves.

TABLE 14

AVERAGE UNIT-DEFORMATION IN CONCRETE; COMPARISON OF EXTERIOR EDGES WITH INTERIOR EDGES OF PANELS

Plus indicates extension and minus indicates shortening

Location of Gage Lines	Gage Lines	Unit-deformation	
		Load 102 lb. per sq. ft.	Load 215 lb. per sq. ft.
Exterior edge of wall panel	565 583	—.00002 +.00107	—.00031 +.00328
Between wall panel and interior panel	546 to 557	—.00007	—.00033
Exterior edge of corner panel	572 577	—.00002 —.00005	—.00016 —.00064
Between corner panel and wall panel	558 to 564	—.00006	—.00041

edge of the panel. The conditions at an exterior edge of the panel are not shown to be more severe than at an interior edge. This fact is of importance in its bearing on the action of a flat slab built without beams at the exterior edges.

42. *Point in Inflection.*—In Group IV, for purposes of detecting the point of inflection, gage lines were placed on the upper and under surfaces of the slab along the line joining the centers of the columns B4 and B3. Fig 64 shows the positions of the gage lines and the variation in unit-deformation along this line. In using the figure it should be

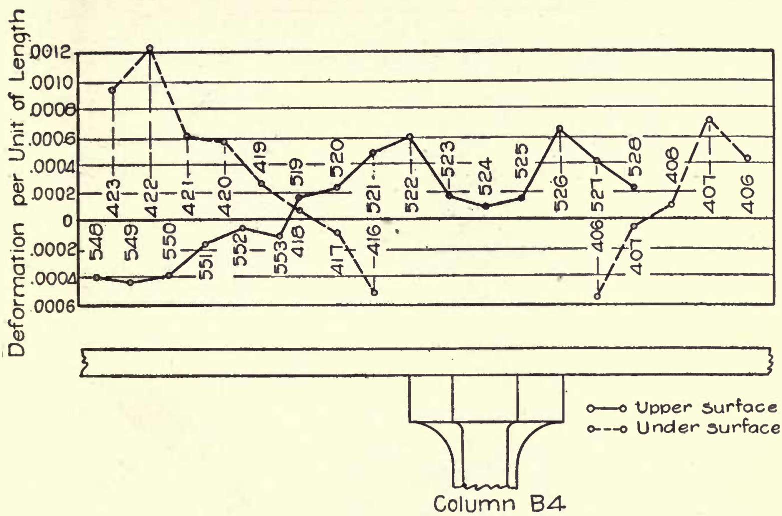


FIG. 64 LOCATION OF POINTS OF ZERO UNIT-DEFORMATION FOR UPPER AND UNDER SURFACES IN GROUP IV OF WORCESTER TEST FLOOR.

borne in mind that settlement of footings probably had much to do with the location of the point of inflection. It appears that the point of zero deformation on the upper side of the slab was farther from the column than that on the under side for both the interior panels and the wall panel. A probable position for the point of inflection, as shown by the intersection of the curves in Fig. 64, is about 33-in. from the column center for both wall panel and interior panels. This corresponds to 17/100 of the panel length from the edge of the column capital and to 20/100 of the panel length from the center of the column.

43. *Locus of Highest Stress.*—In Group IV gage lines were laid out on upper and under surfaces of the slab with a view to determining the locus of the highest stress developed in the elements of the slab parallel to rows of columns. The results for the upper surface are shown in Fig. 65. This diagram indicates that the position of highest stress in a bar crossing the capital is approximately at the edge of the capital. For bars not crossing the capital, the highest stress appears to be at the intersection of the bar with the center line of the row of columns. The locus of highest tensile stress appears to follow rather closely the outline of the capital through 180° and to turn off rather abruptly at the intersection of the capital with the line joining centers of columns. Although it is not possible to get a gage line short enough or close enough to the column to determine what the maximum compressive unit-deformation is, the results of the test indicate that the locus of highest compressive stress is in about the same position as that for tension except that it probably turns off less abruptly at the inter-

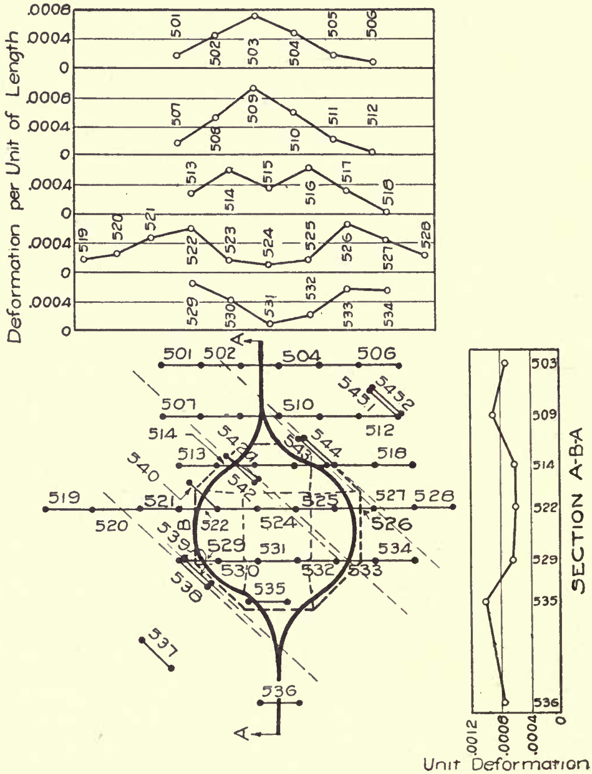


FIG. 65. LOCUS OF POINTS OF HIGHEST STRESS IN A RECTANGULAR BAND OF REINFORCEMENT IN TOP OF SLAB AT COLUMN D4, OF WORCESTER TEST STRUCTURE.

section with the line joining centers of columns. It will be noted that the compressive deformation in gage line 206b is less than that in 206a (Fig. 63). The same phenomenon has been observed in other tests, and is what may be expected.

It may be added that at the load of 215 lb. per sq. ft. the compressive deformations in the section of maximum negative moment were distributed along the section for the full panel width.

44. *Distribution of Stress over Cross Section of Bands.*—Fig. 66 and 67 show the variation of deformation across the width of several bands of reinforcement at sections midway between columns and close to the column capitals.

Some previous tests have indicated that midway between columns the stress distribution in the diagonal bands may be different from that in the rectangular bands, the unit-deformation being greatest in a bar at the edge of the rectangular bands and at the center of the diagonal bands. Fig. 66 indicates that in general the bars in the central por-

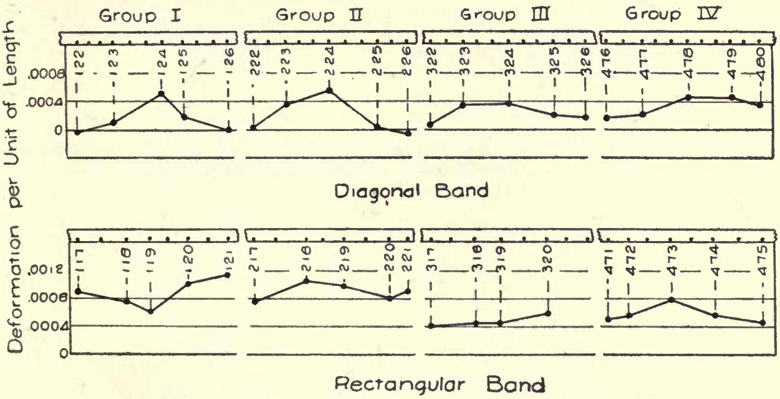


FIG. 66. DISTRIBUTION, MIDWAY BETWEEN COLUMNS, OF DEFORMATION IN DIAGONAL AND RECTANGULAR BANDS OF REINFORCEMENT OF WORCESTER TEST FLOOR FOR LOAD OF 215 LB. PER SQ. FT.

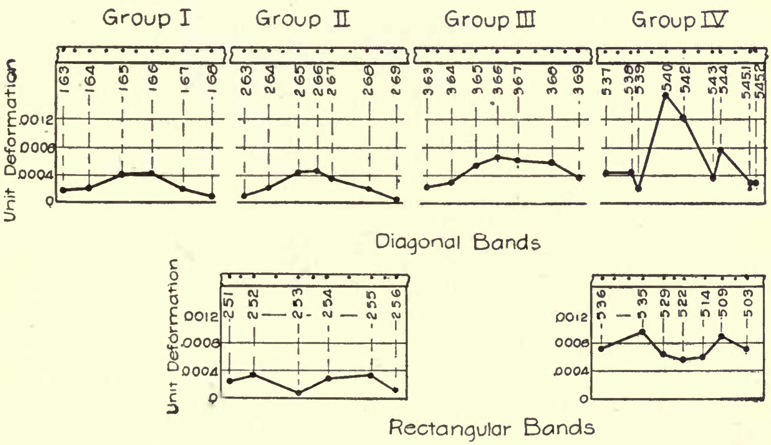


FIG. 67. DISTRIBUTION, NEAR COLUMN CAPITALS, OF DEFORMATION IN DIAGONAL AND RECTANGULAR BANDS OF REINFORCEMENT OF WORCESTER TEST SLAB FOR LOAD OF 215 LB. PER SQ. FT.

tion of the bands show higher stresses than those at the edge. There are one or two cases in which the reverse is true, but there is not sufficient regularity in this to justify the conclusion that this test shows a difference in stress distribution for rectangular and diagonal bands of reinforcement.

45. *Slip of Bars.*—The diagonal reinforcement was lapped at the column capital. The bars of each length extended 12 to 15 in. (32 to 40 diameters) beyond the center line of the column. At several places where the bars were lapped strain gage measurements were taken from one bar across to another for the purpose of determining the

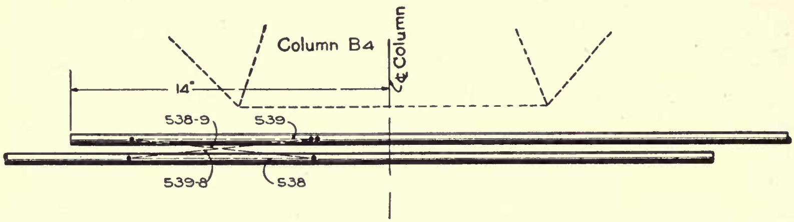


FIG. 68. ARRANGEMENT OF GAGE LINES ON WORCESTER TEST FLOOR FOR MEASUREMENT OF SLIP OF BARS.

amount of slip of the bars relative to each other. Fig. 68 shows the arrangement of gage lines 538, 539, 538-9, and 539-8, used for this purpose, and shows the position of the ends of the bars on which the gage lines were placed. The total change in gage length has been plotted in Fig. 69 as abscissas and the load as ordinates. The movements of the

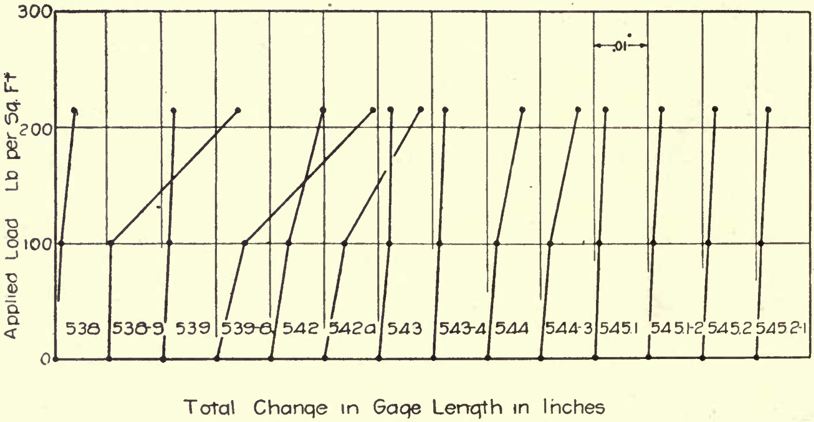


FIG. 69. LOAD-DEFORMATION DIAGRAMS FOR GAGE LINES USED FOR DETERMINING SLIP OF BARS IN WORCESTER TEST FLOOR.

gage points on the two bars relative to each other for loads of 100 and 215 lb. per sq. ft. have been plotted in Fig. 70. In fitting probable curves to the plotted points it was assumed that the curves for the two bars had the same slope at the point where they cross the line joining the centers of the columns A3 and B4 (marked symmetrical axis across bars). This is the same as assuming that the tensile stresses in the two bars are equal at this point, and follows from the symmetry of the arrangement of the bars. If there had been no slip of the bars relative to each other, the curves would touch at the point on the diagram corresponding to the place where the bars cross the line joining the centers of columns A3 and B4. The vertical distance between

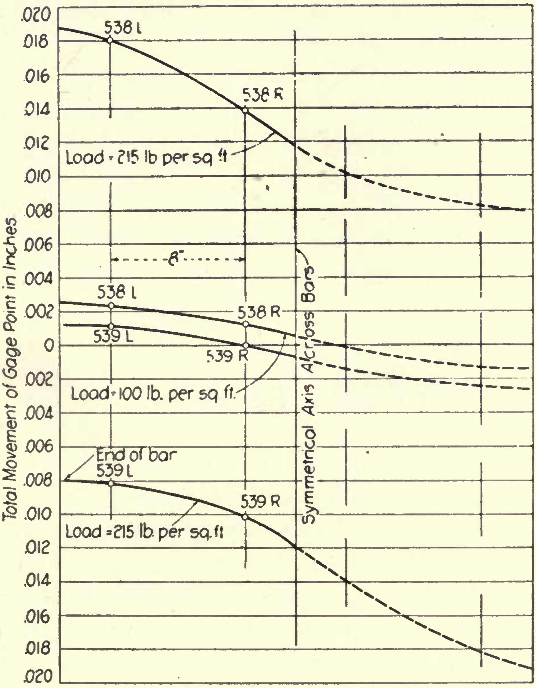


FIG. 70. DIAGRAM INDICATING SLIP OF BARS AT GAGE LINES 538 AND 539 IN WORCESTER TEST FLOOR.

the curves at this point represents the slip of the bars relative to each other.

Assuming that at this place the two bars slipped the same amount relative to a fixed point, but in opposite directions, the zero-line for slip measurements will bisect the vertical distance between curves at their intersection with the symmetrical axis as shown in Fig. 70. Ordinates to points on the two curves indicate the movements of the corresponding points on the bars relative to the line joining columns A3 and B4. Differences between ordinates at any two points on the same curve represent the total deformations taking place in the bar between the points considered. The stress in the bar at any point is proportional to the slope of the curve at the point.

The diagram indicates that at the load of 100 lb. per sq. ft. there was a slip of bars relative to each other of about 0.0013 in. At a load of 215 lb. per sq. ft. the slip of the bars relative to each other had increased to 0.024 in. at the same place. If the two bars did not act in the same way the slip at the place where they cross the center line of the columns may have been all in one bar. This would give a more

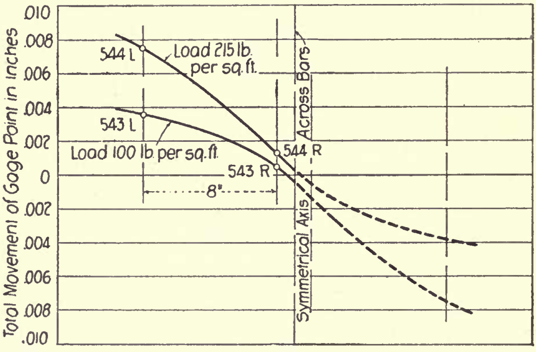


FIG. 71. DIAGRAM INDICATING SLIP OF BARS AT GAGE LINES 533 AND 534 IN WORCESTER TEST FLOOR.

dangerous condition than that in which the slip is the same in both bars at this point.

Fig. 71 was constructed from measurements on gage lines 543, 544, 543-4, and 544-3 in the same manner as Fig. 70 was constructed. It indicates that at a load of 215 lb. per sq. ft. the slip of the bars relative to each other was only about 0.0006 in. It may be due to the smallness of this slip that the difference in stress in gage lines 543 and 544 is so much greater than the difference for gage lines 538 and 539. The difference in stress in the two bars is indicated by the difference in slope of the two curves at the point considered.

From measurements taken in the same way between gage lines 545.1 and 545.2 no slip of the bars relative to each other could be detected at the position of these gage lines, the centers of which were almost on the diagonal passing through the center of the column. At points on the bars near the ends it is likely that there was slip of the bars relative to each other. Measurements on gage lines 542 and 542a indicate at the right hand gage point a slip of one bar past the other of about 0.0078 in. at a load of 215 lb. per sq. ft. Whether this slip extended as far as the center line of the column would be of importance to know, but the observed data do not indicate whether it did or not.

It is not of special importance that these bars showed slip at the ends. This is to be expected if the design includes bars which end in a region of high stress. The significance of the measurements lies in the indications that there was slip at the point where the bar should develop high tensile stress, in the fact that this slip more than doubled with a doubling of the load although the tensile stress in the bar had not reached the yield point, and in the large amount of slip which occurred in some cases. The danger from a small initial slip has been brought out in a series of tests on beams subjected to continued load (*unpublished*).

After the failure of the slab, end slip varying from ½ in. to 1 in. was found in the bars designated by gage lines 539, 542a, and 543, while in gage lines 545.1 and 545.2 no end slip was apparent. It may be of significance that the bars which showed a large amount of slip at failure were the same as those which showed early slip at the point of highest tensile stress, and that the bars (designated by gage lines 545.1 and 545.2), which showed no early slip at the point of highest tensile stress, likewise at the maximum load did not show a slip which was large enough to be observed by the unaided eye.

That the bond stress was high in other bars crossing the column capital is brought out by an inspection of Fig. 64 and 65 which show the varying unit deformations along the lengths of bars. Within a distance of 8 in. (between gage lines 525 and 526) a change in stress of 15,500 lb. per sq. in. took place. This corresponds to a bond stress of 187 lb. per sq. in. if the bond stress be considered to be uniformly distributed. The shape of the curve indicates that the intensity of bond stress at the edge of the capital must have been nearly twice as much as this and it may have been more. This will be well brought out by a consideration of the slopes at various points of any smooth curve which may be passed through this series of points (gage lines 522 to 526 Fig. 64).

46. *Moment Coefficients.*—With the large and irregular settlement of the footings it is not to be expected that the moment coefficients calculated from the observed stresses will have much quantitative value, but a study of the character of the results may be of use.

In a beam or slab loaded in any manner, the total bending moment at any section may be expressed as kWl in which k is a coefficient, W the load on one panel and l the panel length. Values of this coefficient for the positive moment and the negative moment of the reinforcement stresses in Group IV have been computed in the same manner as was done for the Schulze Baking Company Building (see

TABLE 15

CALCULATED MOMENT COEFFICIENTS FOR GROUP IV

Location	Applied Load, lb. per sq. ft.	
	102	215
Positive	.011	.019
Negative	.015	.030
Ratio	.73	.63

article 28). Table 15 gives these coefficients, each value being proportional to the resisting moment of the stresses in one rectangular band of reinforcement plus the components (in a direction parallel to that rectangular band) of the moment of the stresses in one diagonal band. Table 15 shows that these coefficients are much higher for the 215-lb. load than for the 100-lb. load, both around the columns and at sections midway between columns. The difference may be due partly to uneven settlement of footings and partly to partial failure of concrete in tension. In considering the relative values of the negative and the positive resisting moments developed, it should be borne in mind that the slip of the bars at the column would cause a smaller negative bending moment and a larger positive bending moment than would otherwise be the case.

It is seen that the coefficients given in Table 15 are less than those generally used in designing and still less than those found by the more conservative analyses. For the load of 102 lb. per sq. ft. the coefficient of bending moment calculated from the observed stresses was even less than that for 215 lb. per sq. ft. It is probable that even at the higher load the tensile strength of the concrete has played a considerable part.

In Groups I, II, and III, not enough gage lines were read to permit

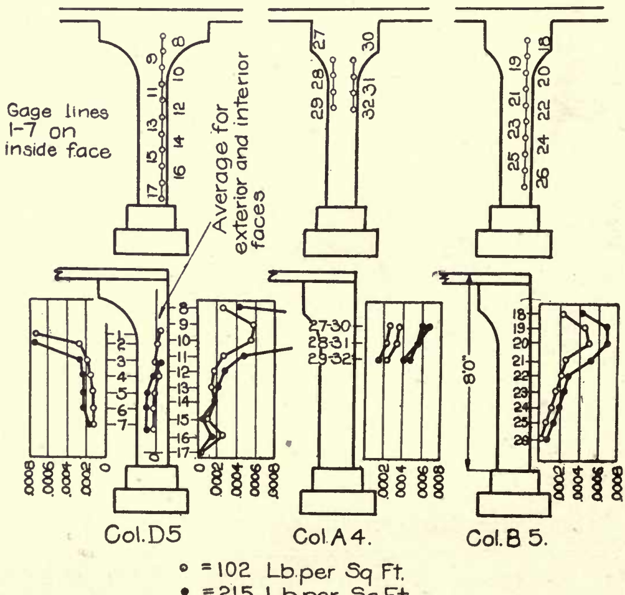


FIG. 72. DIAGRAM SHOWING DEFORMATIONS IN COLUMNS A4, B5, AND D5 OF WORCESTER TEST STRUCTURE.

the calculation of moment coefficients, but it has been shown that the stresses in these groups were lower than those in Group IV.

47. *Deformation in Columns.*—Fig. 72 shows unit-deformations in the reinforcement and in the concrete of columns A4, B5, and D5. These measurements were made for the purpose of detecting bending in the wall columns. The gage lines on the reinforcement were on the bars nearest the outer surface of each column; those on the concrete were on the inner surface of Column D5, that is on the face toward the interior of the panel.

On column D5 (see Fig. 72) deformations were observed on opposite sides of the column. In the lower part of the column where there were no cracks the neutral surface (so far as flexure is concerned) would lie on the center of the column and the average of the unit-deformations measured on opposite sides of the column should be the unit-deformation due to the direct compression. It is found that this average unit-deformation was about 0.00004 at a test load of 215 lb. per sq. ft.

Near the upper end of the column the tension was so large as to

FIG. 73. VIEW OF COLUMNS D5 AND E5 OF WORCESTER TEST-STRUCTURE WITH MAXIMUM LOAD IN POSITION.

cause large cracks (see Fig. 73) and slipping of the outer vertical bars, thus throwing the neutral axis closer to the compression than to the tension side of the column. Averaging the measured deformations for this region of the column, therefore, does not eliminate the flexural deformations. A resultant elongation at the center line of this part of the column should be expected and Fig. 72 indicates that elongation occurred at that place.

For a column fixed at the top of a fixed footing and rigidly connected with the slab at the top of the column but having no lateral movement there, the point of inflection is at a point one-third the column height above the top of the footing. The measurements did not extend down far enough on this column to determine the location of the point of inflection of the column, although the series of gage lines covered 0.93 of the distance from the slab down to the top of the footing. This indicates that there must have been sufficient tilting of the footing to modify the condition of restraint of the column at the top of the footing.

Although no strain gage readings were taken on a corner column, bending of the column at the outer corner of Group IV was observable to the unaided eye at a comparatively early stage in the loading, and at the maximum load bending was pronounced (see Fig. 73). The indications were that bending was more severe in the corner columns than in the columns at the side of a loaded area.

An examination of Table 16 indicates that in Group IV at a load of 102 lb. per sq. ft. the stress in the outer reinforcing bars of the wall columns near the bottom of the capital (12000 lb. per sq. in.) was higher than the average stress in the slab reinforcement where it crossed the edge of the capital of the same wall columns (8000 lb. per sq. in). In Group II, which had larger capitals, no measurements were taken of deformation in the slab reinforcement at a wall column, but the stress in the reinforcement of the wall column, D5, at a load of 102 lb. per sq. ft. was higher (16000 lb. per sq. in.) than that in either wall column of Group IV. However, for the load of 215 lb. per sq. ft. the table indicates that in Group IV the average stress in the slab reinforcement at the wall columns (35000 lb. per sq. in.) had become larger than the stress in the outer reinforcing bars of the columns (19000 lb. per sq. in.). At the same time the stress in the reinforcement of a wall column of Group II (Column D5) had passed the yield point.

In Group II the larger capitals afforded additional stiffness to the slab but very little more to the columns. In this group a very severe

TABLE 16.

STRESSES IN TENSION REINFORCEMENT; COMPARISON OF SLAB AND COLUMN AT WALL.

Location of Gage Lines	Gage Line	Stress in Reinforcement lb. per sq. in.		Average Stress lb. per sq. in.	
		Load 102 lb. per sq. ft.	Load 215 lb. per sq. ft.	Load 102 lb. per sq. ft.	Load 215 lb. per sq. ft.
GROUP IV On Slab at Column A4	567 568 569 570	8100 8300 11200 4500	30600 38800 34200 43200	8000	36700
GROUP IV On Slab at Column B5	579 580 581	8000 9500 8900	44300 37200 41000	8800	40800
GROUP IV On Column A4	27 30	10700 7600	18700 19000	9200	18800
GROUP IV On Column B5	20	14200	20000	14200	20000
GROUP II On Column D5	9	16000	Beyond yield point	16000	Beyond yield point

bending of Column E5 and high bending stresses in D5 followed by complete failure of the latter column occurred before the stress in the slab reinforcement at the central column had reached the yield point. In Group IV no column failed completely, although the bending in the slab was marked, the stress in the slab reinforcement at the central column having reached the yield point.

The results of the investigation of columns are not extensive enough or definite enough to furnish a basis for conclusions on the amount of moment carried by the columns. However, using as a criterion the ultimate loads carried and the manner of failure for Groups II and IV, the behavior of the columns relative to the slab was such as would be expected.

48. *Cracks.*—Fig. 74 gives the location of cracks on the under surface of the slab as shown by an examination at the maximum load and also those found on the accessible portions of the upper surface. The first large crack which was observed was in the bottom of the slab in the outer corner panel of group IV (see Fig. 74). It took approximately the form of a quadrant of a circle having a radius of about 6 ft. with column A5 at the center of the circle. This crack appeared soon after the load of 102 lb. per sq. ft. was applied and it opened rapidly as the loading continued. At a load of 215 lb. per sq. ft. the panel failed with the continued opening of this crack. Probably sev-

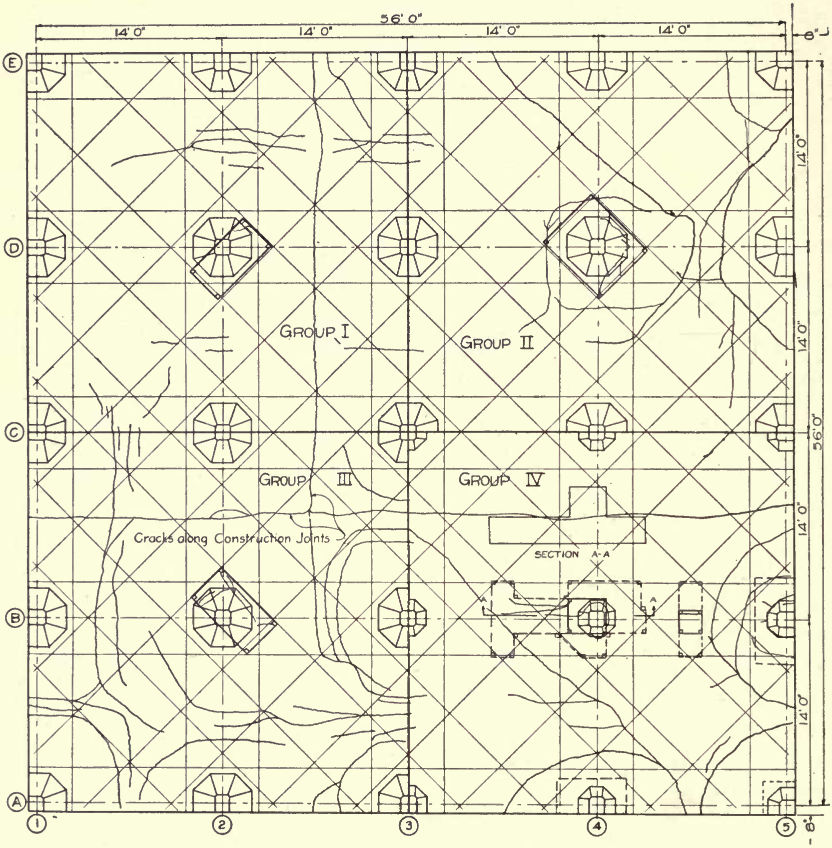


FIG. 74. CRACKS FOUND ON UNDER AND UPPER SURFACES OF WORCESTER TEST FLOOR AT MAXIMUM LOAD.

eral causes contributed to the location of the crack at this place. The unbalanced load caused a bending in Column A5, and it may be expected that this threw the point of maximum positive bending moment outside of the center of the diagonal span. At the short distance from the column at which the crack was located the reinforcement of the diagonal band may not have reached its position in the bottom of the slab. It should be borne in mind that high stresses were observed over the edge of the capital of column A5 in the bars of the diagonal band and that considerable bending of column A5 was visible but that the column did not fail.

In the corner panel of Group II the first prominent crack found took approximately the form of a circle around the corner column

(Column E5) much the same as the crack in Group IV, and a large amount of bending occurred in the corner column, but failure in this group seems to have been due to a weakness in the wall column, D5. In this column the concrete slipped past the upper ends of the vertical reinforcing bars, allowing a crack to form on the outer surface of the column somewhat above the bottom of the column capital, and the column failed in flexure.

The top of the slab was covered with loading material so that the general location of cracks could not be observed. At four interior columns (B2, B4, D2, and D4) the boxes used to give access to gage lines permitted the inspection of the upper surface of the slab over a portion of the column head. Well defined cracks just inside the outline of the capital, as shown within the boxes, were found. At two wall columns (A4 and B5) similar cracks were found.

Uneven settlement may have influenced the formation of cracks. It will be seen from Fig. 74 that many of the cracks are in locations where the reinforcing bars may be expected to be not close to the lower surface. In Group I, having the bars of the rectangular bands carried at the bottom of the slab through to its boundaries, the cracks were comparatively few and small. In this portion of the slab the settlement of the footings was relatively small and regular.

49. *Summary of Results.*—Although settlement of footings puts a serious limitation on the general applicability of the results of the test the following summarized statement of the information obtained is made:

1 In groups having capitals of the same size the variation in stress due to the variation in arrangement and distribution of reinforcement was less, apparently, than that due to uneven settlement of columns.

2 At a load of 102 lb. per sq. ft. the steel stress at the small capital (Group IV) averaged about 50 per cent greater than the stress at the larger capitals. The diameters of the capitals were respectively 0.196 and 0.321 times the panel length. The large stress in Group IV may be due partly to other causes, but it is believed that the small capital is the most important cause.

3 The wall panels and corner panels showed higher steel stresses and generally more pronounced cracks than did the interior panels.

4 This test does not indicate that the wall panels (not the corner panels) suffered because of the absence of wall beams.

5 The point of inflection in Group IV (the group having small capitals) was about two-tenths of the panel length from the center

of the central column, but its exact location is uncertain, since the point of zero unit-deformation on the under surface of the slab was closer to the column than that on the upper surface. The location of the point of inflection probably was influenced by the uneven settlement of the columns.

6 The locus of highest stresses in the bars of a band of reinforcement at a column head followed fairly closely the outline of the column capital through 180°, then branched off and followed the line joining centers of columns. The locus for the compressive stresses on the under side of the slab parallel to a given band occupied a corresponding position so far as may be determined from the data of the test.

7 In few of the bands of reinforcement in which measurements were taken was the stress higher in the bars on the edge of the band than in the central bars. In most cases the stress was highest in the central bars.

8 In cases where bars were lapped as much as 50 diameters beyond the point of maximum stress slipping at that point occurred without the stress having passed the yield-point strength of the steel. The slipping of these bars supports the ruling frequently made that bars should not be spliced at regions of maximum stress. The slipping of bars evidently affected the action of the slab and may have induced failure.

Bars which did not slip were found to have developed a bond stress averaging over the entire gage length 187 lb. per sq. in. At portions of the gage length the bond stress must have been much higher than this.

9 Moment coefficients calculated on the basis of the measured stress in the steel were materially higher at the higher load. Though even at the higher load the coefficients were low, the rapid increase with increased load confirms the view that there is danger in placing reliance on the stresses in the steel measured at ordinary test loads as a basis for determining moment coefficients to be used in design.

10 The bending of corner columns and wall columns was an important feature of the action of the test structure. In certain instances this bending was apparent to the eye.

11 The first large crack on the under surface of the slab was in the corner panel of Group IV near where the diagonal bars were carried from the top of the slab to the bottom.

The location of cracks in the other groups seems to have been influenced by the settlement of the footings, the bending of the outer columns, and the position of the point of carrying the reinforcing bars from the top to the bottom of the slab.

VI. The Test of the Factory Building of the Curtis-Leger Fixture Co.

50. *Description of the Building.*—The floor loaded is in the addition to the Curtis-Leger Fixture Company's factory near Van Buren and Aberdeen Streets, Chicago. This addition is 53 ft. 6 in. by 57 ft. The floor is an 8-in. flat slab of reinforced concrete and is supported on reinforced concrete columns placed 19 ft. and 17 ft. 10 in. apart center to center in the two rectangular directions. It was designed for a live load of 200 lb. per sq. ft. and a dead load of 100 lb. per sq. ft.

A distinctive feature of the Barton Spider Web System of Reinforcement is the use of unit mats as reinforcement for the negative moment at the columns, the mats being independent of the reinforcement for positive moment in the slab. The bars in these mats extend in two directions only, parallel to the sides of the panel. The mats in this building were made up in the shop and consist of ⅝-in. square Havemeyer bars placed as shown in Fig. 75. Certain loose bars were added in this case, and these extend beyond the outlines of the mat. For the positive moment at the center of the panel, four-way reinforcement of ½-in. square Havemeyer bars was used. These bars were not bent up to the top of the slab, but extend along the bottom of the slab

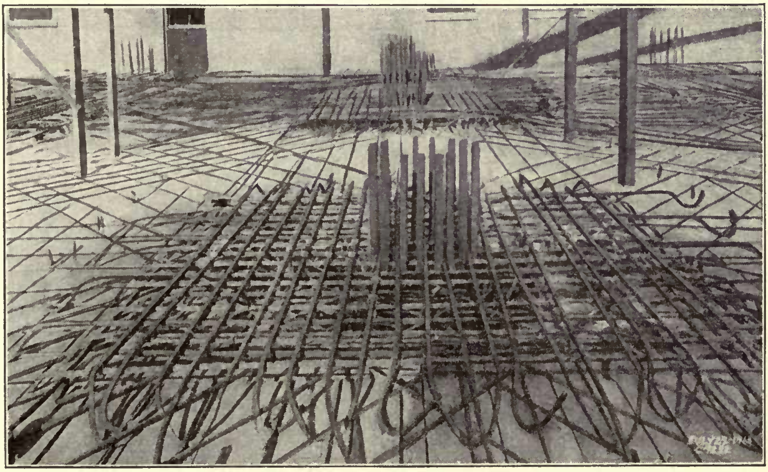


Fig. 75. View Showing Slab Reinforcement in Place in Test Floor of Curtis-Leger Company Building.

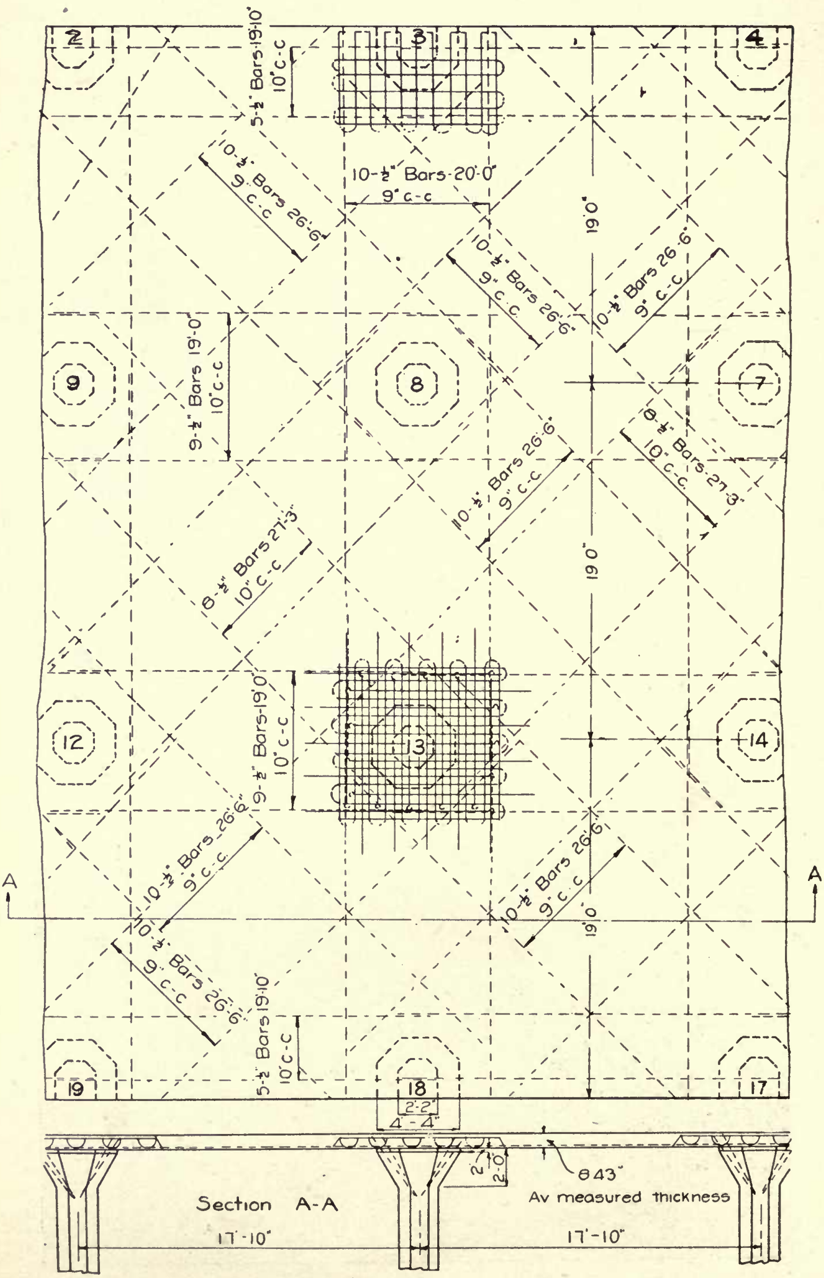


FIG. 76. PLAN SHOWING DIMENSIONS AND REINFORCEMENT OF TEST FLOOR OF CURTIS-LEDGER COMPANY BUILDING.

into the region of compression. They end 6 in. and 7 in. beyond the column center lines in the long and in the short panel lengths, respectively, and 3 in. to 7 in. beyond the column center-line in the diagonal directions. These bars thus serve as reinforcement in compression in the region of the column and in tension at the middle portion of the panel. No drop or depressed head at the column capital was used. The average measured depths to the center of gravity of the reinforcement were 7.04 in. for positions of negative moment and 8.55 in. for positions of positive moment.

The reinforcement for the wall panels is slightly in excess of that for interior panels. Spandrel beams 12 by 22 in. in cross section extend between exterior columns.

The position of the reinforcing bars is shown in Fig. 76.

51. *The Test.*—It was desired to make the amount of load handled as small as possible consistent with developing reasonably high stresses. Instead of covering four entire panels with 200 lb. per sq. ft. and then covering two entire panels with 400 lb. per sq. ft., it was believed to be more satisfactory to use equivalent total areas made up of parts of six panels, as shown in Fig. 77, first with a load of 200 lb. per sq. ft. (boundary dotted in the figure), then rearranging the load to give 400 lb. per sq. ft. over the other parts (boundaries in full line), then adding enough to the latter area to make 500 lb. per sq. ft. This plan gave more complete loading over the principal band of reinforcement and produced negative moments in this region greater than would be obtained under other likely conditions with only two panels loaded. This arrangement of the 500-lb. load also gave easier access to gage lines for the measurement of stress in the vicinity of the columns.

Aside from determining the action of the type of reinforcing, it was expected to obtain information bearing on (a) lateral distribution of stress in bands midway between columns and at the columns and (b) longitudinal distribution of stress along rectangular and diagonal spans. Fig. 78 shows relative location of gage lines on the upper and under surfaces of the test floor.

When the forms were ready and before the concrete had been poured small beveled iron plugs were nailed to the forms in such position that when the forms were removed the smaller base of each plug was in the plane of the under surface of the slab and in proper location for the gage holes used for the measurement of deformation in concrete. Corks were attached to the bars at such places as gage holes in the reinforcement were desired. These steps greatly facilitated the preparation for the test. The concrete was poured on July 30, 1913, and at that time six concrete control cylinders 7½ in. in diameter and 16 in. long were made from concrete taken from the mixer in use on the job. The cylinders were tested at the University of Illinois, November 13 and 14, 1913.

The floor test was made October 22 to 30, 1913. For loading material bags of cement were used. An area equivalent to four panels, shown

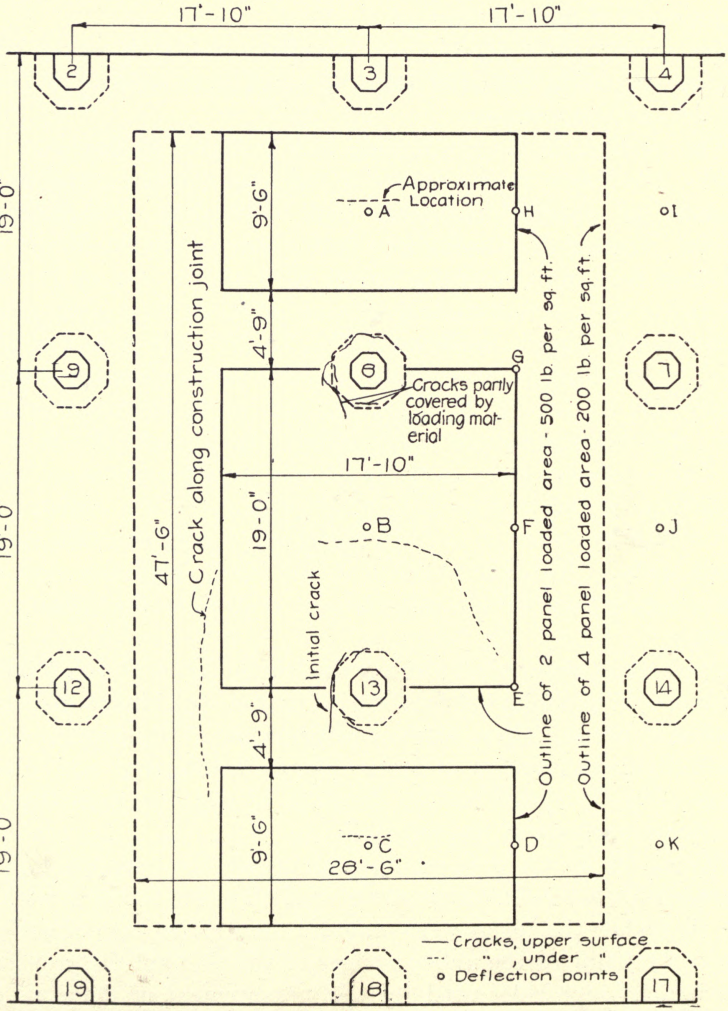


FIG. 77. PLAN INDICATING DISPOSITION OF LOAD AND LOCATION OF DEFLECTION POINTS ON TEST FLOOR OF CURTIS-LEGER COMPANY BUILDING.

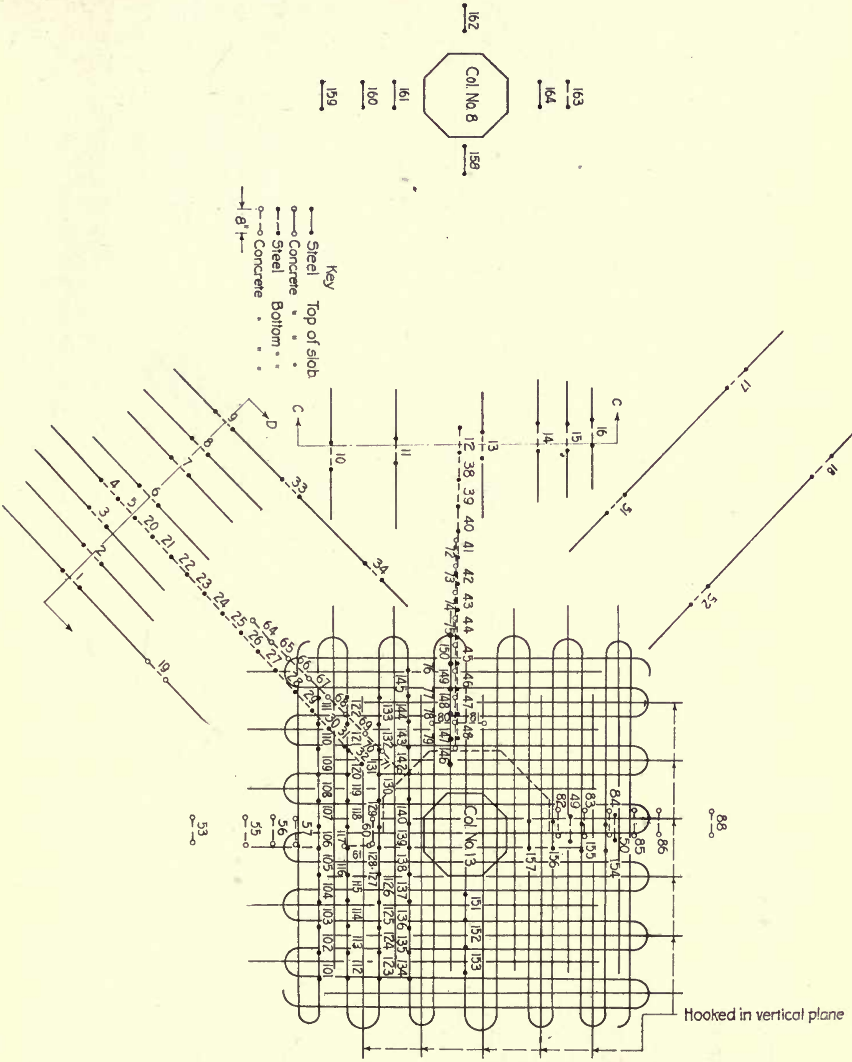


Fig. 78. Location of Gage Lines on Upper and Under Surfaces of Test Floor Curtis-Leger Company Building.

in Fig. 77 was loaded to 200 lb. per sq. ft., the design live load. The load was then removed from one-half of the area, and the total load was concentrated on an area equivalent to two full panels, shown in Fig. 77. Enough additional load was later applied to the two-panel area to bring the average intensity of the applied load up to 500 lb. per sq. ft. over this area of two panels.

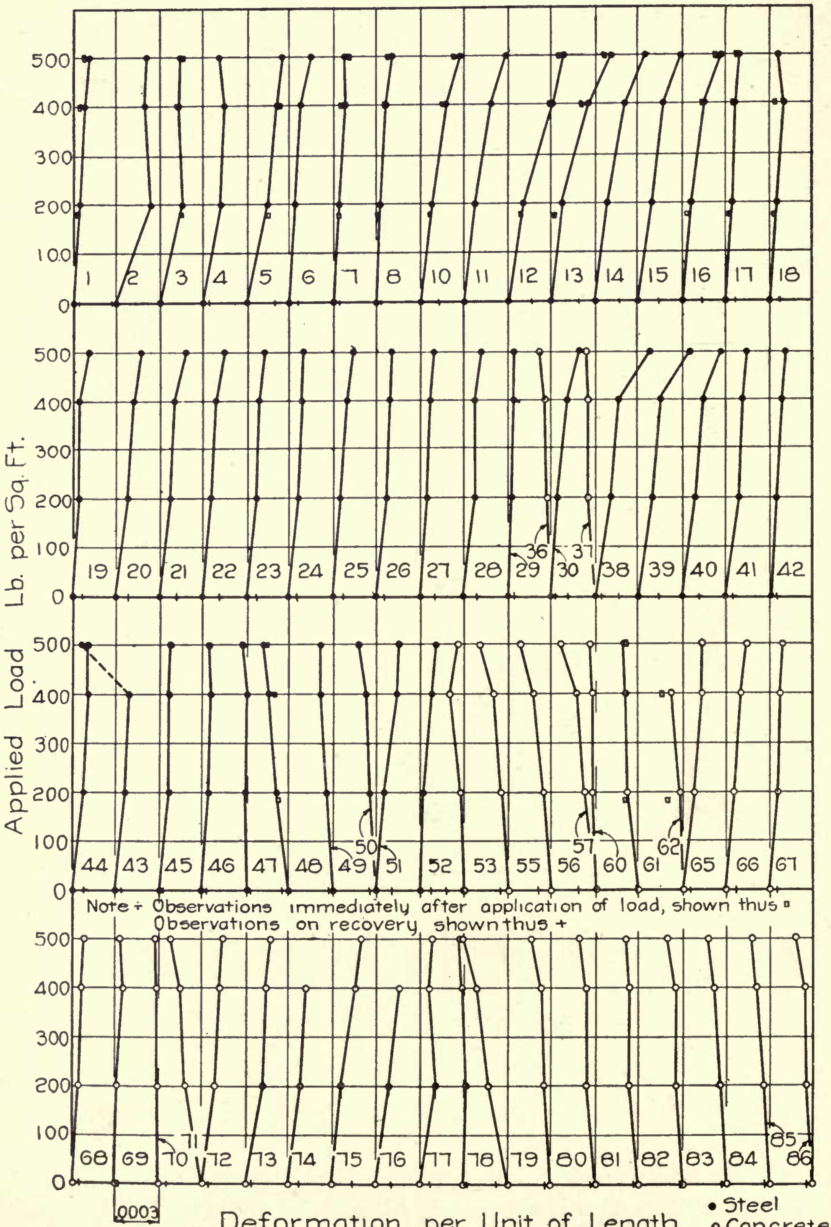


FIG. 79. LOAD-DEFORMATION DIAGRAMS FOR GAGE LINES 1 TO 86 OF TEST FLOOR OF CURTIS-LEGER COMPANY BUILDING.

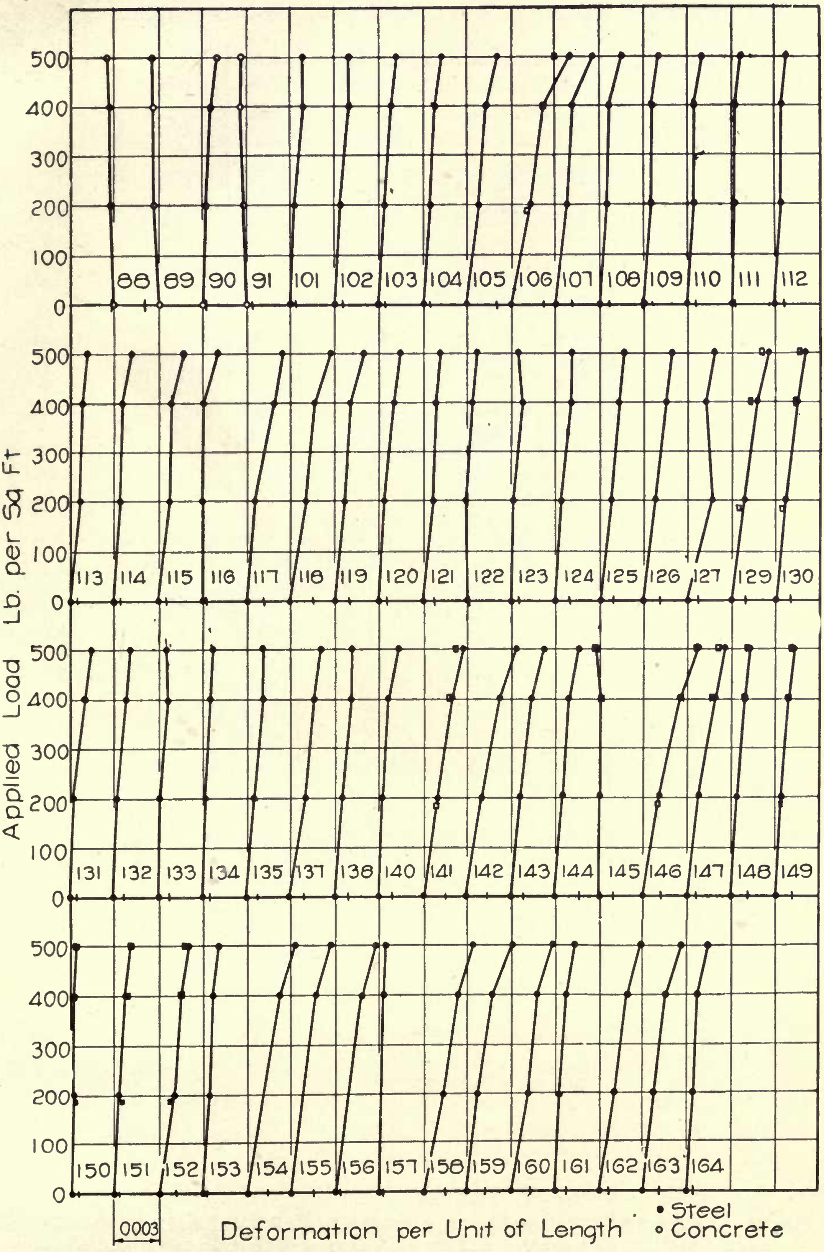


FIG. 80. LOAD-DEFORMATION DIAGRAMS FOR GAGE LLNES 88 TO 164 OF TEST FLOOR OF CURTIS-LEGER COMPANY BUILDING.

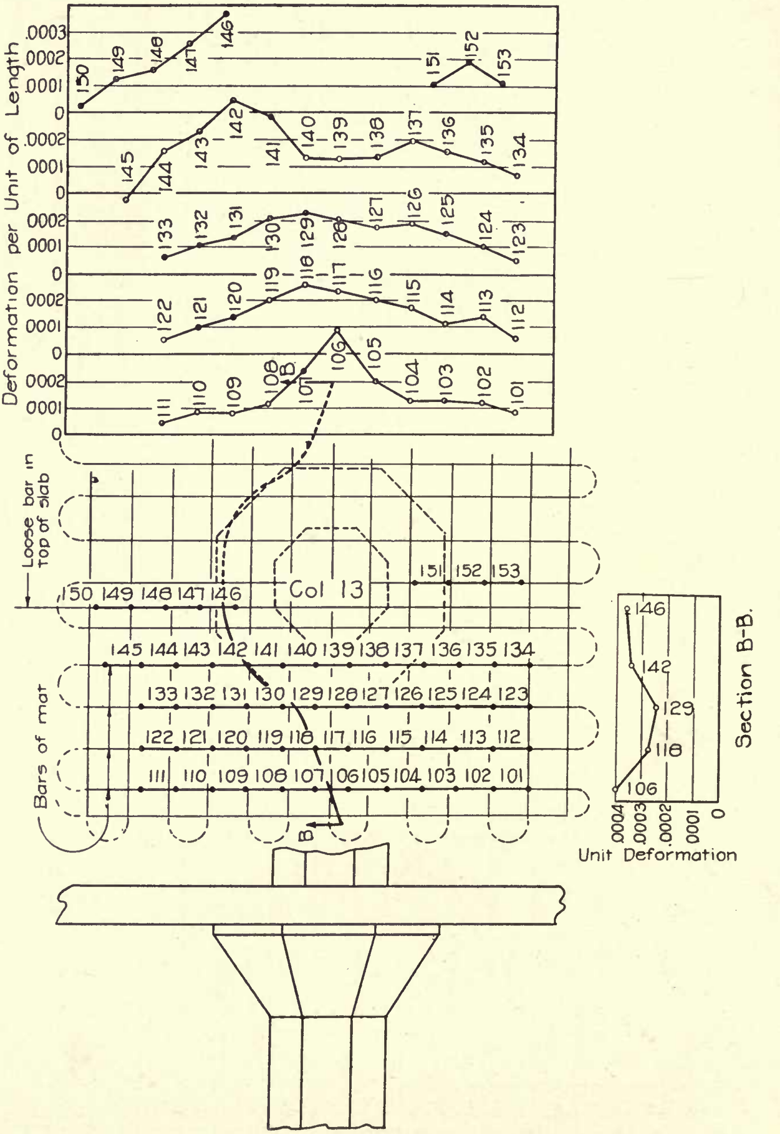


FIG. 81. LOCUS OF POINTS OF HIGHEST STRESS IN REINFORCEMENT IN TOP OF SLAB NEAR COLUMN 13 OF TEST FLOOR OF CURTIS-LEGER COMPANY BUILDING.

Deformation readings were taken on a few gage lines in a number of the most significant positions immediately after completing the loadings of 200 lb., 400 lb., and 500 lb. per sq. ft. When each load had been in place 12 hours or more a complete series of readings was taken. For the load of 500 lb. per sq. ft. the complete series of readings was not begun until the load had been in place 46 hours. When the recovery readings were begun, the greater portion of the load had been off the floor for more than 12 hours, but a small portion had been removed less than three hours before.

52. *Control Cylinders.*—The control cylinders gave strengths ranging from 1230 to 1650 lb. per sq. in. and initial moduli of elasticity ranging from about 2,300,000 to 4,700,000 lb. per sq. in. As a fair working value and one which facilitates comparison with the results of other tests, 3,000,000 lb. per sq. in. has been used as the modulus of elasticity in cases in which it is desirable to interpret unit-deformation into stress.

53. *Tension at Capital.*—Load-deformation diagrams for all gage lines are shown in Fig. 79 and 80. Unit-deformations in the reinforcement which extends in the direction of the 19 ft. side of the panel, found at a floor load of 500 lb. per sq. ft. are plotted in Fig. 81 also. From this the locus of the highest stress has been determined and is shown in this figure as section B-B which, for lack of complete information, assumes that gage lines 154, 155, and 156 (see Fig. 78) lie on this locus. The deformations on this section are shown in the right-hand portion of the figure. The highest deformation on this section corresponds to a stress of 11,000, the lowest to 7,000, and the average to 9,300 lb. per sq. in.

54. *Tension Midway Between Columns.*—Fig. 82 shows the distribution of deformation among bars of the rectangular and diagonal bands of reinforcement midway between columns at the applied load of 500 lb. per sq. ft.

Section C-C for the rectangular band lies midway between columns and is cut at right angles by the long side of the panel. The highest deformation was found in one of the central bars of this band, and corresponds to a stress of 11,500 lb. per sq. in. The average deformation corresponds to 9,100 lb. per sq. in.

Section D-D of Fig. 82 lies normal to the diagonal band at the center of the panel. This section lies near the edge of the area covered by the 500 lb. load, and the deformations here probably were smaller than would have been found if the entire area of this panel had been loaded. This is indicated by the fact that the deformations were

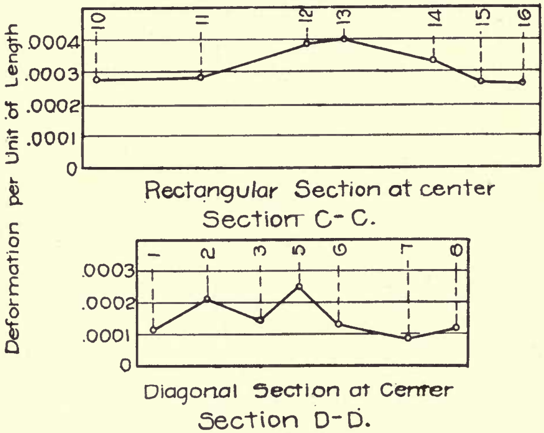


FIG. 82. TENSILE UNIT-DEFORMATION, MIDWAY BETWEEN COLUMNS, IN REINFORCEMENT OF RECTANGULAR AND OF DIAGONAL BANDS IN TEST FLOOR OF CURTIS-LEGER COMPANY BUILDING.

changed only slightly by the shifting of the load from the four-panel area to the two-panel area.

A comparison of the stresses at the gage lines crossed by cracks at construction joints with those at similarly situated gage lines not crossed by cracks shows that considerably larger tensile stress in the reinforcement may be expected at points where the tensile strength of the concrete is impaired than at points where the concrete is intact.

On the short side of the panel no measurements of deformation in the rectangular bands were taken midway between columns because the load was not so placed as to develop representative stresses at that point.

55. *Compression at Column.*—The maximum measured compressive unit-deformation in the concrete was 0.00031. It was found at gage line 79, close to the capital of column 13 on the line of column centers in the direction of the 19-ft. span. The compressive unit-deformation at the corresponding position (gage line 71) on the diagonal line through column centers was 0.00021. If a modulus of elasticity of 3,000,000 lb. per sq. in. be assumed, these unit-deformations correspond to stresses of 930 and 630 lb. per sq. in., respectively. Compressive unit-deformations in these gage lines are shown in Fig. 79, 83, and 84.

To investigate the lateral distribution of compressive stress at the column, measurements of deformation in concrete were taken parallel to the direction of the long side of the panel across the center-line of

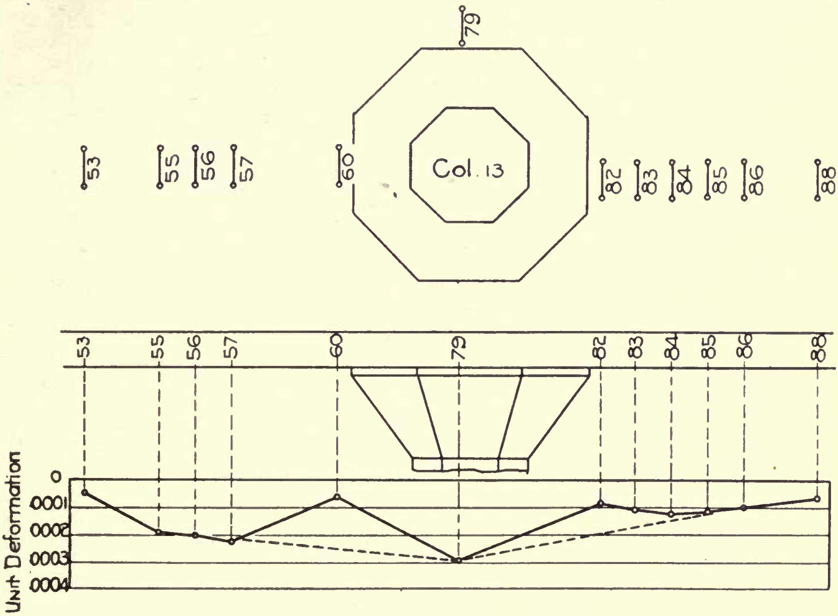


FIG. 83. LATERAL DISTRIBUTION OF COMPRESSIVE DEFORMATION ON BOTTOM OF TEST FLOOR OF CURTIS-LEGER COMPANY BUILDING.

columns, at intervals in the direction of the short side. The extent of area covered by these measurements is shown by the position of gage lines indicated (see Fig. 83). Gage lines 88 and 53 were 84 in. and 76 in. from the center of column 13. The results plotted in Fig. 83 indicate that as far out on either side of column 13 as measurements were taken some compression was developed in a direction normal to the section. If the stress had been uniform and equal to that at gage line 79, a width of about 110 in. would have been required to develop the same total stress as that which was found in the width of about 180 in. It should be noted that gage lines 60 and 82 are close to the column head. By reason of the stiffness of the column head the deformations at these gage lines may not be expected to be as large as may be found farther away from the column head or at points in front or in rear of these positions.

56. *Points of Zero Tension and Zero Compression.*—The manner and amount of variation in deformation along elements of the slab passing over the center of the column in the rectangular and diagonal directions are shown in Fig. 84. The location of the gage lines shown may be found in Fig. 78. An examination of Fig. 84 shows that on the rectangular band the point of zero compression was closer to the

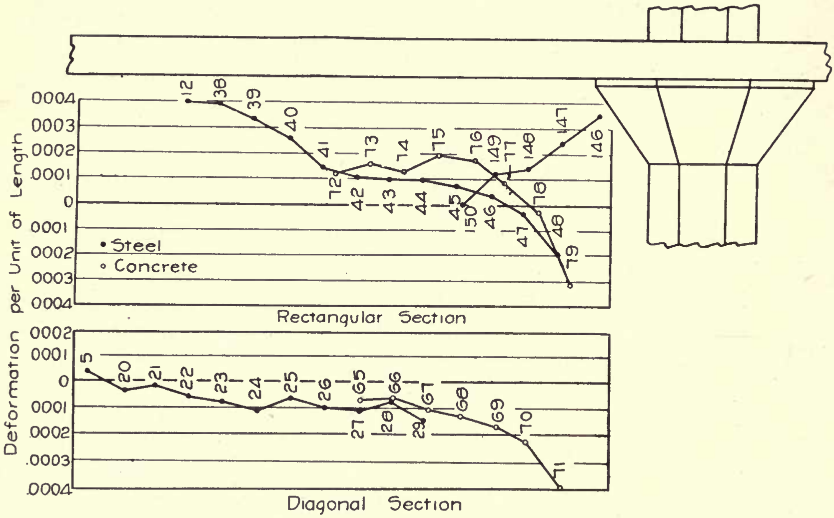


FIG. 84. LOCATION OF POINTS OF ZERO UNIT-DEFORMATION ON UPPER AND UNDER SURFACES OF TEST FLOOR IN CURTIS-LEGER COMPANY BUILDING.

edge of the column capital than was the point of zero tension. No explanation of this is offered. For the diagonal direction, section F-F, the point of zero deformation was found on only the under surface. Table 17 gives the positions of the various points of zero stress.

TABLE 17.

POSITION OF POINT OF ZERO DEFORMATION.

Two panels loaded. Applied load 500 lb. per sq. ft.

Point of Zero Unit-Deformation	Distance from center of column	
	Inches	Ratio to panel length
Bottom; Direct band	42	.184
Bottom; Diagonal band	45	.144
Top; Direct Band, Central Bar	57	.25
Top; Direct Band, other Bars	45	.197
Top; Average, Weighting other bands 3	48	.21

57. *Deflection.*—Load-deflection diagrams for the eleven points shown in Fig. 77 and marked A, B, C, etc., are given in Fig. 85. The maximum deflection at B under the full panel load of 500 lb. per sq. ft. live load was 0.16 in. immediately after the application of this load and 0.17 in. after the load had been in place for 48 hours. These deflections are 1/1425 and 1/1340 of the 19-ft. span.

58. *Cracks.*—In Fig. 77 the locations of cracks found at full load

have been shown by a solid line for the upper surface and by a dotted line for the under surface. These cracks were very fine, the largest being at a construction joint which extended along the direction of the 19-ft. span half way between columns 12 and 13.

59. *Recovery.*—Table 18 gives data on recovery at some of the more important gage lines. This table shows that the recovery was more complete in compression regions than in tension regions whether the measurements were of steel or of concrete deformations. It seems probable that this phenomenon may have been due to the formation of

TABLE 18.

PRINCIPAL FULL–LOAD UNIT–DEFORMATIONS AND AMOUNTS OF RECOVERY.

Load of 500 lb. per sq. ft. applied over two panels.
Plus indicates extension and minus indicates shortening.

Location of Gage Lines	Gage Line	Full-load Deformation	Residual Deformation	Ratio of Residual to Full-load Deformation
On tension reinforcement midway between columns	10	+.00028	+.00012	.62
	11	+.00028	+.00000	.00
	12	+.00039	+.00016	.43
	13	+.00041	+.00020	.50
	14	+.00033	+.00017	.50
	15	+.00027	+.00014	.53
	16	+.00027	+.00013	.50
	Average			.45
On tension reinforcement at capital	106	—.00040	—.00021	.52
	107	—.00025	—.00012	.47
	117	—.00025	—.00012	.47
	129	—.00024	—.00009	.37
	142	—.00035	—.00014	.40
	146	—.00039	—.00012	.31
	Average			.42
On compression reinment at capital	36	—.00008	+.00001	*.13
	37	—.00005	—.00002	*.40
	48	+.00019	+.00005	.27
	49	—.00010	+.00002	.22
	50	+.00012	+.00004	.31
	Average			.27
On concrete at capital	55	+.00020	+.00004	.20
	56	+.00020	+.00009	.45
	57	+.00023	+.00011	.49
	71	+.00021	+.00004	.20
	79	+.00031	+.00008	.25
	80	+.00014	+.00003	.20
	81	+.00012	+.00006	.50
	83	+.00010	+.00001	.10
	84	+.00012	+.00003	.24
	Average			.29

*Not included in average.

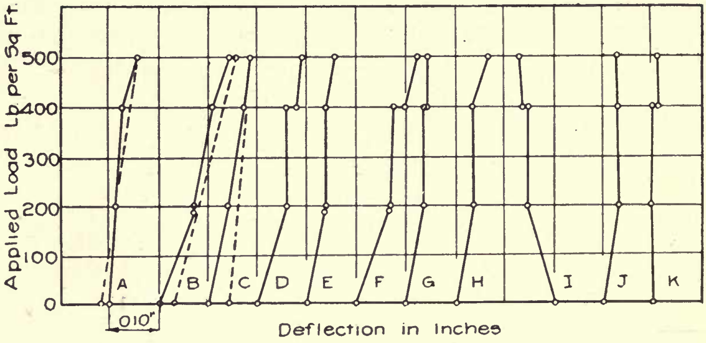


FIG. 85. LOAD-DEFLECTION DIAGRAMS FOR TEST FLOOR OF CURTIS-LEGER COMPANY BUILDING.

cracks, the fractured surfaces of which could not come together again perfectly after the tensile stress was removed.

60. *Summary.*—The following summary is intended to give the main features of the results of the test:

1 With the load of 500 lb. per sq. ft. distributed over an area equal to that of two panels with a view of making its effect in producing stress as severe as possible, the maximum stress in the reinforcement at the column and midway between columns was about 11,000 lb. per sq. in. Calculations made on the basis of design most commonly used in Chicago give a stress of about 25,000 lb. per sq. in. for this load. However, it seems probable that if a larger area had been loaded to the same intensity the stress developed would have been somewhat larger.

2 The highest unit-deformation observed was near the column on the under side of the slab in the concrete and was measured in the direction of the longer span. Based on a modulus of elasticity for the concrete of 3,000,000 lb. per sq. in. this unit-deformation corresponds to a stress of 930 lb. per sq. in.

3 The point of zero unit-deformation on the under surface of the slab was closer to the column than that on the upper surface. For this reason the location of the point of inflection is not known with certainty, but the indications are that it was at a distance of about two-tenths of the panel length from the center of the column.

4 The deflection under twice the design live load plus the dead load was about 1/1400 of the span.

5 The cracks were very small, the largest being along a construction joint. The stresses in gage lines crossing this crack were enough larger than the tension in similar gage lines not crossing a

crack to indicate that the tensile strength of the concrete adds considerably to the resistance of the slab at this load.

6 The recovery was more complete in regions of compression than it was in regions of tension regardless of whether the measurements were taken on concrete or on steel.

VII. General Comments.

61. *General Comments.*—As was remarked at the beginning, the circumstances surrounding the floor test of a building are unfavorable to securing definite and uniform quantitative results. The distribution of the resistance of the structure to parts beyond the portion which is loaded and the effect of the tensile strength of the concrete, greatly modify the action of the structure. The physical conditions connected with the tests are unfavorable to securing exactness. Conclusions drawn from such tests must be of a general nature and must be confined to the general behavior of the structure. The following comments are given:

1 The stresses measured in the reinforcing steel were relatively low. It is felt that the values of these stresses should not be taken as representative of the stresses which may be developed in the structure when it is loaded over a large area for a considerable time. That this view is not inconsistent with the general practice in designing reinforced concrete may be seen by examining laboratory tests of reinforced concrete beams which have percentages of reinforcement comparable with those in the flat slabs tested. In such beams measured stresses of 5,000 to 20,000 lb. per sq. in. in the steel account for only one-fourth to one-half of the external bending moment. In the tests of flat slabs there is no indication that the tensile resistance of the concrete contributes less to the apparent strength of the structure than is the case with beam construction. It is evident that attention must be given to the mechanics of the structure in determining the requirements for making designs.

There is difficulty in evaluating the compressive deformations of the concrete in terms of stress, since the modulus of elasticity of the concrete in the slab may not agree with the values determined from test specimens. The observations on compression are useful in finding the distribution of compressive stresses.

2 For negative moment the locus of maximum stress in a direction perpendicular to a panel edge was a line which followed the column capital for nearly 180° and merged into the panel edge a little distance away from the column capital. In the Schulze Baking Company

Building the measurements were of compression on the under side of the depressed head. In the Worcester test structure and in the Curtis-Leger Building the measurements were made on the tension reinforcement.

3 In the Shredded Wheat Factory the tensile stresses resisting negative moment across the panel edge would average as high at locations intermediate between columns as exist at points close to the column. It is apparent that the actual distribution along a section of negative bending moment would be affected by the size and spacing of the bars crossing the section.

In the Schulze Baking Company Building also it appears that bars across a panel line at a location midway between columns developed resistance to negative bending moment.

Information having a bearing on the distribution of tensile stresses across panel lines was not obtained in any other test discussed in this bulletin.

4 In the building in which there were depressed heads around the column capitals and in which information was obtained on the distribution of the compressive stresses over the section of maximum negative moment, the Schulze Baking Company Building, there were indications that the compressive stresses of the negative moment were taken almost entirely in the portion of the section within the width of the depressed head and that there was very little compression in the thin portion of the section between depressed heads. In the Worcester slab, which had no depressed heads, at the load of 215 lb. per sq. ft. the compressive stresses in the section of maximum negative moment were distributed along the section for the full width of the panel, although the stress midway between columns was less than that closer to the column. In the Curtis-Leger Building compressive stresses were found in the section of maximum negative moment as far away from the column capital as measurements were taken.

5 In the building having a relatively large thickness of depressed head, the Schulze Baking Company Building, the compressive stresses on the under side of the thin portion of the slab close to the depressed head and perpendicular to its edge were nearly as large as those in the same direction on the depressed head close to the column capital.

6 An increase in the deformations in the section of maximum positive moment was found when the loaded area was changed from a group of panels to a row of panels. This change of loading was made in the Shredded Wheat Factory and in the Soo Terminal. How much of the increase may have been due to a proportionally smaller contri-

bution by the tensile resistance of the concrete is not known, but it is evident that the positive moment must have been increased considerably by this change in loading.

7 High bending deformations, due to eccentric loading, were found in columns located at edges of loaded areas. In the Shredded Wheat Factory, a severe bending moment in a column of the basement story was observed when panels of the first floor on one side of this column were loaded. Even with nine panels loaded bending deformations were found in interior columns, evidently due to difference in the slab moments on the two sides of the column. In this case the bending was in a direction opposite to that found when the column was at the edge of the loaded area. In the Soo Terminal, a one-story structure, marked bending phenomena were observed in columns at the edge of the loaded area, and tensile deformations were found of such amount that even considering the compression due to dead load the tensile resistance of the concrete must have been exceeded. The position of the point of inflection of the elastic curve of flexure was in fair agreement with the usual analysis. In the Schulze Baking Company Building, the bending of columns at edges of the loaded area was an important feature of the action of the structure in the test, the largest bending apparently occurring in a column at a corner of the loaded area. In the Worcester Slab Test, the bending of certain wall columns and corner columns was apparent to the eye, and large tensile deformations were observed in the column reinforcement. Although the bending action was not different from that which may be obtained by analysis, it seems well to call attention to the phenomena observed, since provision for resisting the bending moment produced by the eccentric loading of columns (both wall columns and interior columns) may be overlooked by some designers.

8 In the one building in which load was applied to a wall panel having a lintel beam, the Shredded Wheat Factory, diagonal cracks were found on the interior side of the lintel beam near its ends. None was found on the outside of the beam. The cracks extended upward and away from the ends of the beam. The phenomenon was probably the result of the twisting action produced by bending moment developed in the slab at its edge by the load on the wall panel and transmitted to the lintel beam through the monolithic connection between the slab and the beam.

9 In the two one-story structures tested, the Soo Terminal and the Worcester test structure, the unevenness of settlement of the footings was sufficient to interfere with interpretation of the results. In

a building of several stories the rigidity of the structure may be expected to cause it to settle more as a unit. It is evident that in a one-story structure unusual precautions should be taken to guard against uneven settlement.

10 The tests which have given most definite results and results most useful for comparison with analytical treatment have been made on slabs whose thickness was small in relation to the span.

11 Progress in obtaining experimental knowledge of flat slab structures may best be made through a series of tests on structures designed solely for test purposes and planned systematically to bring out the fundamental differences between different types of design and the effect of varying certain elements of design. Occasional tests of floors may give interesting information, but the differences in design and construction among the different structures may be so unsystematic as to make the results not comparable, rendering them useful mainly for judging of workmanship and the sufficiency of the design.